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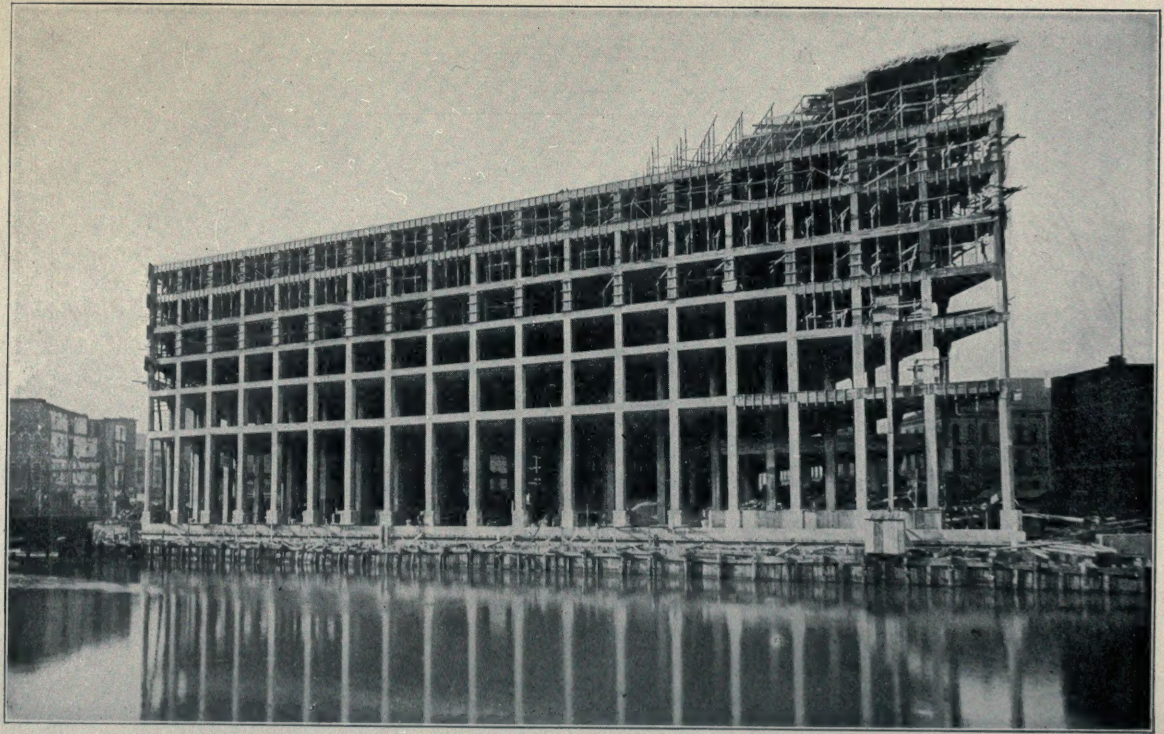

Bostwick-Braun Hardware Co's. Bldg. (Waterfront) Toledo Ohio. Under Construction. "Mushroom System."
A. Bentley & Sons, Contractors. G. S. Mills, Architect. C. A. P. Turner, Engineer.

CONCRETE STEEL CONSTRUCTION

PART I.—BUILDINGS

A PRACTICAL TREATISE FOR THE CONSTRUCTOR AND THOSE COMMERCIALLY ENGAGED IN THE INDUSTRY

BY

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BRIDGES, BUILDINGS AND MANUFACTURING PLANTS

CONCRETE STEEL CONSTRUCTION, Etc.

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PREFACE

There is unfortunately today a much too popular notion that concrete is mere mud, a conglomerate mixture of sand, stone and cement which any chump ought to understand fully. How far this notion is from being correct is best understood by those who have worked carefully from day to day, year in and year out in this line, and realize fully the breadth of the field.

That this notion has indeed possessed many in the engineering and architectural profession is evident by the ridiculous presumption of those totally innocent of any practical experience in this line of business in accepting appointment on committees supposed to draw up safe rules for the conduct of this class of construction resulting in dangerous regulations, loss of life to workmen and increased cost to the owner for inferior construction.

While making a business of putting up reinforced concrete construction and not of writing books, the author is moved to make an effort in this line in view of the fact that present treatises on this subject contain no discussion of safe and unsafe details worthy of the name, no formulae for figuring the strength of multiple way systems that would indicate that the authors ever have built and tested a building and absolutely no suggestion as to the possibility of figuring the elastic behavior of the slabs or their strength with a degree of precision within several hundred per cent.

Another feature in which the majority of treatises are of little value lies in the lack of information as to the cost of executing many kinds of work.

As no human work is perfect, the writer will merely offer his contribution on the subject as an attempt to fill, as best he may, what he considers a want in the field. If the writer's effort in this line tends to prevent loss of life in erecting a class of construction which, barring gross ignorance of its design, is unquestionably the safest to erect, most permanent, fire proof and satisfactory known, his aim and object will be attained.

In the attainment of this object the writer's efforts will be directed to the description only of methods which he considers

best, to the criticism of common errors, without waste of time in discussion of freak theories or types of construction practically unknown in the commercial field, to further spare the time of the reader by stating in the fewest possible words, facts determined in executing work rather than furnishing long-winded, theoretical or mathematical disquisitions which have perhaps some bearing on the subject matter only in the imagination of the author.

Another reason for attempting a treatise on concrete-steel building work, the writer will confess, is, that in his business dealings with building Commissioners, Inspectors, City architects, etc., he has found them good fellows all, but apparently hailing from the remote counties of Missouri in their extreme "having to be shown" proclivities.

Thus he has been forced from business exigencies to conduct a kind of free school on the rudimentary principles of mechanics and horse sense as applied to reinforced concrete construction and stand the expense of railroad fare at a thousand miles per trip with time and hotel bills thrown in. At that, some of his less brilliant scholars would frequently find something contrary to his teaching in some treatise, and, because it was printed in a book, think it must be SO. Then if facts presented did not accord with what was printed it was regarded quite likely as very detrimental to the facts.

Thus it seems desirable to him to have some of the facts put in a book so that they might appear in the "must be SO" class, and further that he might incidentally have the satisfaction of charging for valuable information heretofore furnished gratis by business necessity and at a large cost for the privilege of so doing.

C. A. P. TURNER.

Minneapolis, June, 1909.

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CONCRETE STEEL CONSTRUCTION

CHAPTER I.

Introductory.

The history of structural engineering as a science dates from the early part only of the last century. The progress made has been remarkable indeed, and the materials mainly used have varied during well-defined periods. Up to 1860 timber and cast iron were mainly used; from 1860 to 1890 wrought iron with some cast iron, was generally employed in bridges and other engineering structures; from 1890 to the present time steel has replaced wrought iron; and while, for long-span bridges, it will perhaps be some time before a more suitable metal is found, yet for short spans, buildings, warehouses and the like, the enterprise of the American manufacturers of Portland cement has placed at the disposal of the engineer a new material, reliable, if properly handled, and of reasonable cost, which bids fair to *largely supplant* steel in the construction of minor engineering works. Indeed today, a warehouse designed for a capacity of 400 pounds per square foot of floor, columns 16 to 24 feet centers, can be built more cheaply of reinforced concrete than of wood frame and floor with similar brick walls. Where the strength required is less, timber at the present rate, is slightly cheaper, since the cost of centering, for light and heavy construction is the same. Still, the difference is so slight that, considering saving in insurance, owners will shortly realize that they cannot afford to continue the construction of fire traps if they are to realize the maximum profit on their investment.

In discussion of concrete-steel construction we must consider, first, the action of concrete with steel, the function of each in the combination, the problems presented by beams, slabs and columns separately, and, finally, the mixture of concrete and questions of cost in convenient placing of the reinforcement.

The strength of Portland concrete in compression is equal to that of our best building stone, with the advantage that it can

be placed in a monolithic mass. The tensile strength like stone, is greatly inferior to that in compression. The concrete yields but little—the stretch being confined to a weak section. When, however, steel is imbedded in the concrete and properly disseminated through it, experience shows that the deformation is at least ten times as great before fracture. In the tests by some American investigators, the concrete beams do not seem to fill the above conditions and the age of specimens was insufficient from which to draw reliable conclusions.

In short, the condition leading to the combination of concrete and steel in a beam or girder is this: the concrete is an excellent and trustworthy material for compression and steel for tension, hence, steel should be distributed in such manner as to carry the tensile chord strain and tensile web stress. To do this economically we can reason by analogy with a truss or beam. The further from the neutral axis the more effective the unit section, hence the reinforcement for tensile chord stress should be at the bottom of the beam or as close to it as satisfactory protection against heat of fire will admit. Now the beams in a building are of constant section, and since a continuous beam is stiffer and stronger than a beam of the same section discontinuous over supports, the ideal concrete-steel beam should be continuous and the top flange reinforced over supports.

Concrete-steel construction is capable generally of as exact mathematical analysis as timber frame, and it should not be employed blindly, but carefully figured by an engineer conversant with the theory of flexure. The writer has no fine-spun theories to present which endeavor to take into consideration the tensile strength of the cement, but merely the suggestion that it is conservative to disregard it entirely and figure on the steel alone.

Before taking up in detail, discussion of the combination of concrete and steel, it seems in order to turn our attention to the concrete and the materials entering into it, their characteristics, value and fitness, and proper proportions to use.

Cement.

Portland Cement only should be used in a reinforced concrete frame or structure.

The following is a specification adopted by the American Society of Civil Engineers:

ARTICLE 2.

Portland Cement.

Definition. This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3 per cent has been made subsequent to calcination.

Specific Gravity.

The specific gravity of the cement, thoroughly dried at 100 degrees C., shall be not less than 3.10.

Fineness.

It shall leave by weight a residue of not more than 8 per cent on the No. 100, and not more than 25 per cent on the No. 200 sieve.

Time of Setting.

It shall develop initial set in not less than thirty minutes, but must develop hard set in not less than one hour, nor more than ten hours.

Tensile Strength.

The minimum requirements for tensile strength for briquettes one inch square in section shall be within the following limits, and shall show no retrogression in strength within the periods specified:*

Age.	*Neat Cement.*	*Strength.*
24	hours in moist air	150-200 lbs.
7	days (1 day in moist air, 6 days in water)	450-550 "
28	days (1 day in moist air, 27 days in water)	550-650 "
	One Part Cement, Three Parts Sand.	
7	days (1 day in moist air, 6 days in water)	150-200 lbs.
28	days (1 day in moist air, 27 days in water)	200-300 "

*For example the minimum requirement for the twenty-four hour neat cement test should be some specified value within the limits of 150 and 200 pounds, and so on for each period stated.

Constancy of Volume.

Pats of neat cement about three inches in diameter, one-half inch thick at the centre, and tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(*a*) A pat is then kept in air at normal temperature and observed at intervals for at least 28 days.

(b) Another pat is kept in water maintained as near 70 degrees F. as practicable, and observed at intervals for at least 28 days.

(c) A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for five hours.

These pats, to satisfactorily pass the requirements, shall remain firm and hard and show no signs of distortion, checkng, cracking or disintegrating.

Sulphuric Acid and Magnesia.

The cement shall not contain more than 1.75 per cent of anhydrous sulphuric acid (SO_3), nor more than 4 per cent of magnesia (MgO).

ARTICLE 3.

Accelerated Test.

The object is to develop those qualities which tend to destroy the strength and durability of a cement. As it is highly essential to determine such qualities at once, tests of this character are for the most part made in a very short time, and are known, therefore, as accelerated tests. Failure is revealed by cracking, checking, swelling, or disintegration, or all of these phenomena. A cement which remains perfectly sound is said to be of *Constant Volume.*

Failure to meet the requirements of the accelerated test in shipments direct from mill need not be sufficient ground for rejection. The cement may be held for twenty-eight days and a retest made at the end of that period. Failure to meet the requirements at this time should be considered sufficient cause for rejection.

The accelerated test is a rough and ready means for determining without elaborate equipment whether cement is fit to use. Cement known to have been stored by a dealer for some time should be promptly rejected if it fails in this test.

A rough and ready test for activity may be made by making small blocks of one cement to two of sand, mixing in warm water and storing in a warm place noting how they have hardened by scratching with a knife and breaking in the hand with a hammer

after a week or ten days, etc. A little experience enables the detection of a questionable cement.

The contractor in the line of concrete work should train his foremen to provide proper protection for all cement brought to the job. Dampness from insufficient protection will render the cement lumpy and while it may not destroy its setting properties it will greatly reduce its sand carrying powers and efficiency or may even render it entirely worthless.

ARTICLE 4.

Uniformity of Portland Cement.

Manufacturers of nearly all brands of Portland cement are using their best efforts and sparing no expense to turn out a product which is as uniform as possible. As a result, the statement is justified that among those brands which have been on the market for a period of five years, there is a greater degree of uniformity in Portland cement than in the merchant steel on the market today and that by the exercise of ordinary care there is no risk from the standpoint of lack of uniformity in this material for structural purposes where the material is handled in the manner usual and proper in putting up concrete steel construction.

ARTICLE 5.

SPECIFICATION FOR MATERIAL.

Sand.

Sand used should be clean and coarse or a mixture of coarse and fine grains with coarse grains predominating, which should be free from clay, loam, mica and other impurities.

Test for Sand.

In order to determine the amount of clay, dirt or other impurities, a simple, practical test is to take an ordinary glass quart preserve jar, put in two-thirds of a pint of the sand with water and put on the cap. Shake thoroughly and allow it to settle. The result will be that the coarser grains will go to the bottom in the order of their size, and the silt and light impurities will settle in a layer at the top, giving the observer a means to gage the extent of the impurities accurately and the character of sand in point of the proportion of coarse, medium and fine grains in its

make-up. From three and one-half to four per cent of clay in the form of finely divided silt will do no harm in a bank sand or gravel for reinforced concrete work. Even higher percentages than this have been claimed to increase the strength of the concrete under test, though where it is exposed to the elements and the action of frost a percentage even as high as this seems to be quite detrimental. However, in building work, which is usually under cover, it does no harm whatever.

Gravel.

Gravel where used should be composed of clean hard pebbles and sand free from clay and other injurious foreign matter such as rotten stone, hardened lumps of clay and the like. A sample having the coarser material screened out may be tested for impurities in the same manner as that given for the sand.

Broken Stone.

Broken stone used shall consist of sound crushed stone such as trap rock lime stone, granite, hard sandstone or conglomerate. If the texture of the stone is crystalline and there are no portions of rotten stone or hardened clay as is sometimes found in oölitic lime stone and shale the crusher-run may be used, eliminating a portion of the sand which would otherwise be used in the mix.

If however, the stone under the hammer can be readily reduced to a fine palpable powder as is the case with some shales and the type of lime stone referred to, the dust should be entirely removed.

It is better where possible to use only that stone which is found durable when exposed to the action of the elements and frost and the harder the stone the stronger the concrete that can be made, using it as an aggregate.

ARTICLE 6.

Proportions of Materials.

In concrete steel building construction the proportions which experience indicates most economical in concrete for slab and beam construction, columns and footings except where the loads to be carried are unusually great is one part of cement to two parts sand and four parts broken stone or gravel, this being indicated by the expression 1-2-4.

These proportions are customarily taken by measure, each

bag of cement being estimated as equal to one cubic foot in volume, thus the proportions of 1-2-4 mean one sack of cement, two cubic feet of sand and four cubic feet of crushed stone.

The size of the stone for reinforced concrete work in ordinary building construction should range from one inch down, with the observance for screening as outlined under the specifications for stone.

The first requirement in proportioning concrete for reinforced work is to see that there is an excess of the fine material over and above that required to fill the voids in the coarse aggregate. The volume of voids in the coarse aggregate are greater with a uniform size of stone than when the sizes of the coarser aggregate vary from coarse to fine, and for that reason the writer prefers the crusher-run of stone where the stone is either granite, trap or hard crystalline stone.

ARTICLE 7.

Analysis of Strength of Concrete.

Concrete may be defined as an artificial conglomerate stone in which the coarse aggregate or space filler (generally a hard natural stone, furnace slag or pebble) is held together by a cement matrix. Having selected a given coarse aggregate, the strength of the concrete depends on the strength of the mortar matrix, in other words, on the ratio of cement to sand in the mortar for all samples of the same age, formed under the same conditions.

The Strength of the Concrete Depends then—

First, on the grade of sand and the proportion of the cement to the sand in the mortar;

Second, upon the hardness and the character of the coarse aggregate;

Third, on manipulation and the conditions under which the concrete is cured or hardened;

Fourth, on the age of the specimen.

The mortar made with a very fine sand is only about half as strong as that made with coarse and medium grains and for that reason the specification regarding the character of the sand should be given careful attention.

As shown by Feret, quite a variation in the proportion of medium, coarse and fine grains of sand will give nearly the same

strength so that the average clean coarse bank sand will generally fill the requirements for a good concrete mortar.

The richer the mortar the stronger the concrete. As noted above, the writer recommends a one-to-two mortar for reinforced concrete and where high working stresses are to be used in reinforced concrete columns a mortar in the concrete as rich as one cement to one and one-half sand with an increase of twenty-five per cent in the working stress.

Coarse Aggregate.

The effect of the strength of the coarse aggregate upon the strength of the concrete, in tests of concrete made with shale rock crushed from 1½ inch down, at Duluth, show the shale concrete about sixty-five to seventy per cent as strong as trap rock concrete and the trap rock concrete from ninety to ninety-five per cent as strong as that made with lake gravel for the coarse aggregate. These tests, as the writer recollects it, were made on concrete about four months old.

ARTICLE 8.

Manipulation and Conditions of Curing.

While the quality of the cement, sand and aggregate have more or less influence on the resulting concrete we may say that with any good brand of first class Portland Cement, clean coarse sand and hard crushed stone, substantially the same results will be secured under identical conditions of mixing and curing. The latter conditions have a most decided influence on the strength of the concrete. Whether sufficient water has been used to permit and promote perfect crystallization of the cement, whether an excess amount of water has been used and the fine and coarse materials have been allowed to separate or segregate, whether the concrete has been thoroughly mixed and whether the conditions of curing were favorable, such as keeping the concrete damp and preventing it from drying out too rapidly or whether it has hardened under unfavorable conditions of frosty weather. On this account it is difficult to harmonize the large number of isolated tests that have been made by independent investigators under widely varying conditions.

In building work, however, it is a fortunate fact that except in cold weather where the work requires special treatment the

general conditions for hardening are most favorable. After one floor has been put up the next is erected thereon within a week or such a matter and the excess water dropping from the upper floors keeps the concrete in the lower properly wet rendering the conditions of hardening and curing far more favorable than those of the ordinary laboratory test.

ARTICLE 9.

Increase in Strength of Concrete With Age.

The following table shows compressive strength of concrete as determined by test made at the Watertown Arsenal in 1899. 1-2-4 mixture.

Brand of cement.	7 days.	1 month.	3 months.	6 months.
Atlas	1,387	2,428	2,966	3,953
Alpha	904	2,420	3,123	4,411
Germania	2,219	2,642	3,082	3,643
Alsen	1,592	2,269	2,608	3,612
Average	1,525	2,440	2,944	3,904

The above gives a fair idea of the increase in strength of concrete with age.

After the period of six months the concrete in ordinary building is found to increase slowly in strength and considerably in hardness and rigidity. Thus it seems that the stiffness of a long span slab will increase about twenty per cent between two months and twelve to fifteen months and the strength perhaps in a lesser ratio in view of the fact that the compression element only in the combination is hardening and increasing in strength.

ARTICLE 10.

Coefficient of Expansion.

The coefficient of expansion of concrete is practically that of mild steel. Some investigators have made this coefficient per degree of Fahrenheit a trifle under and others a trifle over .0000065 which is usually accepted for mild steel, hence there is no injury to the composite material by ordinary changes of temperatures.

ARTICLE 11.

Bond Between Concrete and Steel.

In the design of any combination of concrete with steel the bond between the two elements is of importance. Concrete setting in the air shrinks and grips the reinforcing members with a vice like grip. The richer the mixture the greater this shrinkage stress and the better the bond. In concrete setting in water this shrinkage is lacking and in this position deformed reinforcement or mechanical bond is desirable.

Those familiar with technical literature have unquestionably noted that most of the failures of reinforced concrete have occured where deformed bars have been used. This, as the author looks at it, is due to the fact that the average advocate of the deformed bar seems to place so much stress upon the magical efficiency of the fin corrugation or bulb on the bar, that the poor contractor who buys them is sometimes led to believe that this remarkable deformity will bond together properly the sand and the stone in the concrete, and hence there is little if any need of cement and proper design or workmanship as no adhesion is required, with the natural result that ninety per cent of the failures occur with this class of reinforcement.

With a suitable design and properly arranged reinforcement the writer has never had occasion in the course of his experience to figure upon the bond value between the two materials as it is amply provided for where due precautions have been taken to render the design safe to execute by properly tying the material together with suitable reinforcement and the use of the necessary cement.

ARTICLE 12.

Reinforcing Steel.

Steel for reinforcement should be tough, homogeneous metal, preferably medium steel, from sixty to seventy thousand pounds for ultimate strength, having an elastic limit of not less than half the ultimate strength the percentage of the elongation equaling twenty-two per cent in eight inches.

Unfortunately a large amount of brittle rerolled rails and other low grade metal has been advertised and sold as concrete reinforcing material. In use of this the writer has seen bars

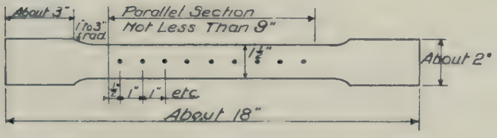
which would break in handling with the slightest jar or shock and such material is not calculated to inspire confidence on the part of the contractor who guarantees the test capacity of the work.

Standard Specifications for Medium Steel.

Ultimate strength, 60,000 to 70,000 pounds per square inch. Elastic limit, not less than one-half the ultimate strength. Percentage of elongation 1,400,000 divided by the ultimate strength.

*Bending test, 180 degrees to a diameter equal to thickness of piece tested, without fracture on outside of bent portion.

Test Pieces.



Pieces to be of same thickness as the plate.

All tests and inspections shall be made at place of manufacture prior to shipment.

The tensile strength, limit of elasticity and ductility shall be determined from a standard test piece cut from the finished material. The standard shape of the test piece for sheared plates shall be as shown by the above sketch.

On test cut from other material the test piece may be either the same as for plates, or it may be planed or turned parallel throughout its entire length. The elongation shall be measured on an original length of 8 inches, except in rounds of ⅝ inch or less in diameter, in which case the elongation shall be measured in a length equal to eight times the diameter of section tested. Two test pieces shall be taken from each melt or blow of finished material, one for tension and one for bending.

Material which is to be used without annealing or further treatment is to be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated

*The bending test is most practical in investigating quality of steel on the work.

before use, the specimen representing such material is to be similarly treated before testing.

Every finished piece of steel shall be stamped with the blow or melt number, and steel for pins shall have the blow or melt number stamped on the ends. Rivet and lacing steel, and small pieces for pin plates and stiffeners, may be shipped in bundles securely wired together, with the blow or melt number on a metal tag attached.

Finished bars must be free from injurious seams, flaws or cracks, and have a workmanlike finish.

ARTICLE 13.

Machine Mixing.

Concrete for the concrete steel building should be machine mixed, preferably in a batch mixer. Some of the continuous mixers do good work where bank gravel is used as the aggregate and fail where crushed stone is used. The writer prefers such a batch mixer as the Smith, Cube, Polygonal or Ransome, which may be charged with cement, sand and stone by measure and the exact amount of water added. The water content in the mix is better supplied for a large piece of work by a tank which will contain the amount of water needed for a batch arranged with the usual float trap valve so that all the operator needs to do is to pull the string and the tank of water is discharged at once into the mixer. This insures a mixture of regular consistency and results in a material saving of time.

Where the work is of sufficient magnitude to permit an overhead hopper into which the sand and stone may be elevated and discharged by gravity into the mixer as desired a large saving in labor results. Where the mixing plant is near a track the hopper may be filled from the cars by a derrick and suitable clam. Where the aggregate is brought to the building by team load a platform arranged so that the wagon may be driven over it and the stone or sand dumped thereon and then elevated and discharged into the top of the hopper is about as economical an arrangement as the writer has seen.

A view of a mixing plant of this kind used in the erection of the Lindeke-Warner building of St. Paul, erected by Butler Bros., is shown in the figure (a.)

In figure (b) is shown the mixing plant used in the erection

Fig. a.

of the John Deere Plow Company's building in Omaha, the hopper in this case being charged by the locomotive crane, using the clam for transferring the materials from the cars to the hopper

ARTICLE 14.

Consistency of Concrete.

For building construction and reinforced concrete work generally it is necessary that the concrete shall be mixed so that it will flow slowly and thoroughly surround the reinforcement but it should be no more plastic than is required to attain this result. If mixed too dry and tamping is depended upon voids will be left around the steel and the face of the concrete when the forms are removed, will be found rough and full of pockets and the

Fig.b. Side view, under construction, showing handling of materials by locomotive Crane. Concrete work erected by Leonard Construction Co. of Chicago, at rate of one story per week.

work will present an appearance of weakness which it very likely does not possess.

If on the other hand too much water is added there is liable to be a certain amount of segregation and separation of the material in the mix which will leave weak concrete in different places. In placing the concrete columns should be filled first then beams and finally the slabs, the operation being continuous as far as possible. If an attempt is made to reverse this program and fill the beam before the adjacent column is filled the concrete will flow in an inclined direction to the column and as each batch is deposited and washes over the inclined surface the light inert matter, cement and fine sand will be washed down into the column and an inferior concrete and one of little strength will be found at the bottom on removal of the column forms.

Where to Make Joints in the Work and How to do it.

Splicing in beams and slabs should preferably be made in the center and should be vertical. The reason for this is that where the concrete is allowed to flow out on an inclined plane in the beam the inert material known as laitance comes to the surface.

preventing a good bond when the new concrete is added. In fact the writer has seen instances where a wedge shaped piece of concrete three feet long and running from two inches in thickness to one quarter inch at the end has dropped away from the beam due to this manner of placing, the bond being insufficient to carry the weight of the piece.

The remedy is to break up the surface of the old concrete or thoroughly clean it with a wire brush and grout it with a neat cement before proceeding to cast the new work.

Before going further into the practical details of executing work we will now turn our attention to the distinctive general types of construction and the theoretical computation of the strength of beams, slabs, columns and the various details or systems in common use and the strong and weak points of the various details of construction.

CHAPTER II.

General Types of Concrete Steel Construction.

ARTICLE I.

Classification.

As we review the history of all types of structural work we find the engineer or designer influenced in his first efforts with a new type by the forms of construction he has been previously accustomed to use. Thus as wrought iron began to replace timber for railroad trestles the longitudinal bracing was identical with that used in timber construction; indeed, at first whether these braces ought not to be of timber from fear of the unknown dangers that might result from the unequal expansion of these braces if iron and the ground on which the trestle was founded was gravely considered and today not a few of our concrete theorists are deeply concerned regarding equally insignificant questions.

The earliest type of timber construction has been followed or imitated closely by some of the pioneers in concrete steel construction and also in not a few of our buildings even today. This type may be described as columns supporting main girders joist spanning from girder to girder and a thin floor covering the joist.

Type I. figure one, shows the interior of the Busch Model factory and is of this type.

Type II. figure two, shows the interior of the Manufacturers Furniture Exchange, Chicago, and is typical of that class of concrete structures that imitate or follow the type of timber construction known as mill buildings. It differs from type I of figure one in an attempt to reduce the cost of centering by using girders in one direction and spanning from girder to girder with a slab supported on two sides by the girders.

Type III, figure three, shows the evolution of a distinctive concrete type beam or ribs in two directions and slabs supported on four sides.

Type IV, figure four shows a second distinctive concrete type in which the centering has been simplified to the maximum extent and the elements involved are two only columns and continuous flat slabs supported directly by the columns.

Types I and II are amenable to similar theoretical treatment applied in the past to timber and steel construction. Type III is partially so, requiring distinctive treatment for the slab. Type IV represents a true monolith possible only in concrete steel construction and requiring theoretical treatment differing radically from that applicable to types I and II.

These four types we will now consider from the following standpoints:

1, Safety,

2, Economy,

3, Ease and accuracy of computation,

4, Fire resisting qualities.

From the standpoint of safety in erection economy and ease and certainty of computation type IV ranks first.

Type III ranks next in economy, equal in safety if the beams are made continuous and second in ease and certainty of computation.

Type II ranks third in economy, second in ease of computation and is most dangerous during construction.

Type I ranks fourth in economy, second in safety during erection and third in work of computation.

The above rating as to safety is based on the general record of these types. It should be noted, however, that all can be executed with a reasonable degree of safety if special attention is paid to features of design which will be discussed later. As to the rating from the economic standpoint this is a mere matter of computation given of course the proper mathematical basis for the same and the student or contractor is advised to make his own figures rather than to accept the statement of the author.

From the fireproof standpoint evidently the form which exposes the least area to heat, which presents the most uniform distribution of metal to provide for the temperature stresses resulting from unequal heating will rank first. On this basis then the four types in their order of merit rank in reverse order, IV first, III second, II third and I fourth.

In the above types we have the following problems of computation in design.

1, Beams, simple, continuous, partially continuous, etc.

2, Slabs supported on two sides,

3, Slabs supported on four sides,

Type I. Fig 1.

Type II. Fig. 2.

Type III. Fig. 3.

Type IV. Fig. 4.

4, Slabs supported on four points or corners.

In each of the slab problems we also must consider the condition of the ends as in considering beams.

5, Columns practically similar for all types.

ARTICLE 2.

Computation of Beams.

As noted in the introduction we propose to disregard the value of the concrete in tension and this being the case there is no excuse for an attempt to split hairs or compute the strength nearer than three or four per cent.

The three laws of mechanics governing the internal distribution of stress at any section of any beam may be stated as follows:

Sum of tensile stresses = Sum of compressive stresses

Resisting shear = Vertical shear

Resisting moment = Bending moment.

Demonstration of these three laws will be found in any of the standard works on mechanics of materials and an especially clear proof in the work on this subject by Mansfield Merriman.

It will be noted from the first law that the sum of the tensile stresses equals the sum of the compressive stresses and from the fact that we are figuring on the basis of the steel in the bottom of a beam, taking the entire tensile stress we have at once that the resisting moment equals the stress on the steel multiplied by the distance from the steel to the center of gravity of the compressive forces acting in the top of portion of the beam. If the concrete is thoroughly rigid and hard these compressive forces will vary from the top of the beam towards its neutral axis in magnitude directly as their distance from the neutral axis and the line of action of their resultant would evidently be one third of the distance between the top of the beam and its neutral axis.

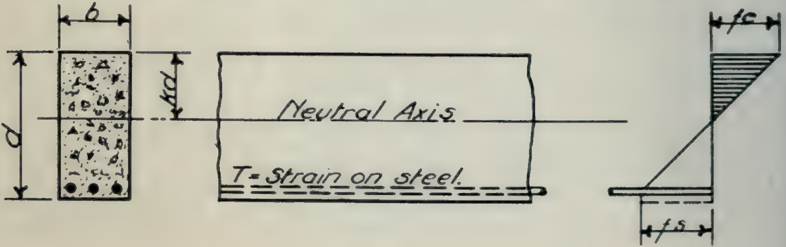


Fig. 5.

Let the depth of the beam figure 5, above the center of the steel equal d and the position of the neutral axis equal the distance kd below the top of the beam. Let M equal the moment of resistance Then the resisting moment

$M = T\ (d - \frac{1}{3}kd)$ (T equaling the tensile stress on the steel)

This proportion of the depth ($d - \frac{1}{3}kd$) does not vary greatly for a variation in the percentage of the reinforcement of from three-quarters to one and one-quarter per cent for the same grade of concrete.

Test by Prof. Talbot taking Ec at 12 as the value for concrete of a 1-2-4 mix nine months old would give the values from 85 to .87d. Now bearing in mind the fact that the modulus of elasticity of concrete varies as the load exceeds one third of its ultimate strength as it seems to do with steel it becomes evident that for all practical purposes between these limits we may treat the effective depth of the beam as .85d.

For slabs where the reinforcement is under one-half of one per cent we may take the effective depth as .9d for one way reinforcement.

While this method of computation has been commonly used among many engineers making a specialty of concrete steel construction the credit for bringing it prominently before the general public is due to Capt. John S. Sewell, proceedings of the American Society of Civil Engineers, 1906.

He writes the formula:

$M = h\ d\ a\ b\ t\ s$,

"h" is a constant,

"d" is the distance from the extreme element in compression to the axis of the steel.

"a" is the sectional area of steel per inch of width.

"b" is the width of a rectangular beam in inches.

"ts" is the elastic limit of steel.

In this formula it may be stated that "ts" does not indicate the full ultimate strength but rather the point at which large cracks commence to develop. Substituting for "ts" in this formula the working stress on the steel we have a simple practical formula which may be safely used for all work of this character providing the percentage of steel is limited to such values as will not result in an excessive working strain on the concrete.

The percentage of steel should not exceed one and one-half per cent of the area of the concrete.

The mathematical investigations of beams through all percentage of reinforcement possess more value from the academic than from the practical standpoint and as the purpose and object of this book is to place before the contractor a practical work on this subject no discussion of this character will be presented.

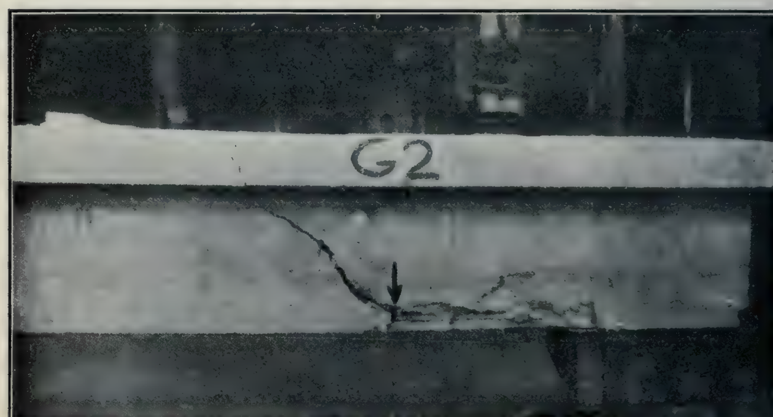


Figure 6. Showing Beam Failure diagonally between center and support.

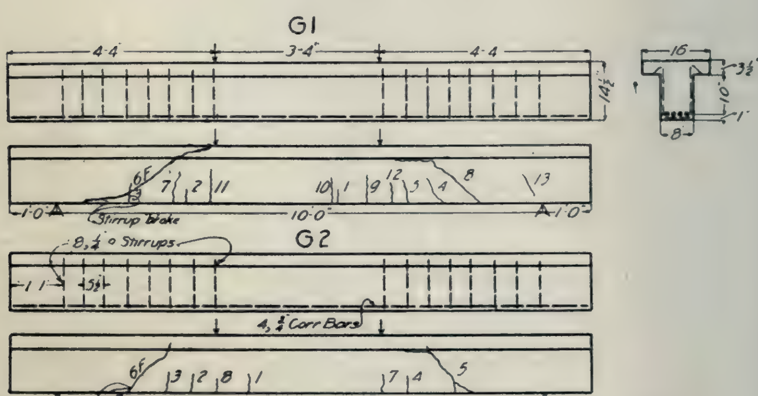


Fig. 7. Beams of Series F— Reinforcement and Cracks.

ARTICLE III.

Problem of the Tensile Stress in the Web of Concrete Steel Beams.

Having derived a fairly accurate and scientific formula for the maximum tensile range of stress in the beam we have now to consider the exceedingly knotty problem of the tensile web stress in a beam.

Unfortunately in a composite material like reinforced concrete this becomes an exceedingly complicated problem in molecular physics for which thus far no mathematician has succeeded in offering even the semblance of a scientific solution. The best that has been offered to date is the suggestion of an analogy to some type of truss such as a Warren, Howe, or Bollman, each of which will be treated under special details.

Reference to figures from pages 94, 95 and 133 of the Bulletin of the University of Wisconsin No. 197, indicate clearly that the failure of a simple beam is more likely to occur between the end and the center than at the center.

Since from the engineering standpoint we need to figure the weakest section of a piece of construction and we have at best only an indirect analogy to work upon the theory of the simple beam in reinforced concrete must be characterized as in a crude and unscientific condition and its design is too often governed by the mere whim or notion of the designer rather than by exact principles of mechanics.

Types of construction I and II then embodying the primitive idea of producing a structure or floor by the combination of simple beams may be condemned as unscientific from the practical standpoint that the strength of the weakest section cannot be determined mathematically in a general manner at the present state of the art by the exact principles of mechanics and that the most important features are left to the judgment, whim or caprice of the designer.

The numerous failures with type II accompanied by loss of life to the workmen add force to this criticism. Fortunately these types are not distinctive or natural concrete types of construction and it is a fact that in the latter the tensile web stresses diminish in relative intensity so that their consideration becomes merely a matter of secondary rather than of primary importance

and our admitted ignorance of the exact laws of mechanics applicable thereto does not menace the safety of the construction in the latter types.

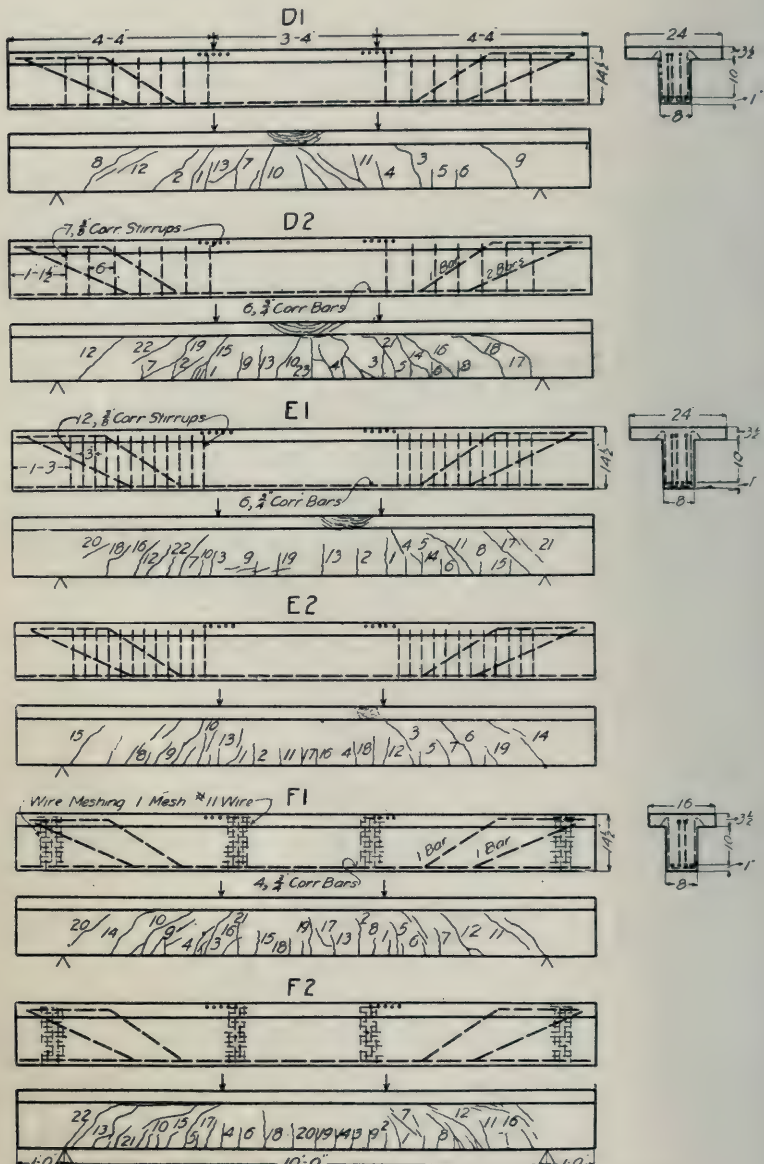


Fig. 8. Beams of Series F—Reinforcement and Cracks

CHAPTER III.

Computation.

ARTICLE I.

General Formulae.

Before proceeding to the derivation of general formulae it is desirable to call attention to the general principle governing the placing of the reinforcing metal.

Any concrete slab or beam supported at intervals either at points or on walls if loaded deflects or bends and if the load is excessive cracks the concrete first from the under or tension side upward in a plane practically normal to lines joining the supports. Hence effective reinforcement should cross these planes of natural weakness at right angles.

Since the steel reinforcing bars can act only by strain along their length we see that in a general manner whether they are in single layers running in one direction or in double or multiple layers running in two or more directions that as far as the steel is concerned that their action must be similar in character to the flanges of a beam and hence since the strength of the plate, regardless of the distribution of the stresses in the concrete must depend on the tensile stresses in the steel, the general laws of bending would be of identically similar form to those for a beam.

Notation.

Let M_x = the external moment at any section distant from the support at the point of greatest strain.

M_I = the internal moment of resistance for the same section.

W = the total load on the beam or slab taken for convenience in 1,000 pound units.

L = the span in feet.

d = the distance from the top of the concrete to the center of steel in inches.

A_s = the area of one reinforcing rod in square inches.

f_s = the unit strain on the steel per square inch in 1,000 pound units.

Σ = the usual sign of summation.

B = a coefficient which may be variable or constant in value to be determined and applied in the following discussion to formulae for bending moment.

D = a similar coefficient applied to deflection formulae.

Δ = the deflection for any load.

Then by statics and the laws of beams we have the following general equations:

(1) $M_X = M_I$

(2) $M_X = (B)\ W\ L = \frac{1}{12} \Sigma\ A_S \times .85d \times f_S = M_I$

(3) $\Delta = (D) \dfrac{W\ L^3}{\Sigma\ A_S\ d^2}$

(1) Follows from the elementary principles of statics. (2) Follows from the usual law of beams that the external moment is a coefficient times W the total load times L the span which equals the internal moment of resistance as we have shown for bending and equals the area of the steel times its tensile stress times its effective lever arm.

Deflections following the laws of beams equal a coefficient times $\frac{W\ L^3}{E\ I}$ The E and $(.85)^2$ we will take care of in the coefficient to be determined. The I is the moment of resistance $\Sigma\ A_S\ (.85d)^2$ as shown in our treament of the simple beam.

ARTICLE 2.

Application to Mushroom System.

Having now established our general formulae let us proceed to the application of the formulae to the simplest possible case of reinforced concrete construction shown in figure 9. This type is known as the Mushroom System. It consists of four belts of rods crossing the panel from column to column and spread out over the supplementary cantilever reinforcement at the top of the column shown in figure 9.

The red areas in the figure show the areas where the rods are only one layer or belt in thickness. Evidently we would select the point of greatest weakness in the center red area in a direct line between the columns. A practical test of the construction up to the yield point of the steel after the concrete is

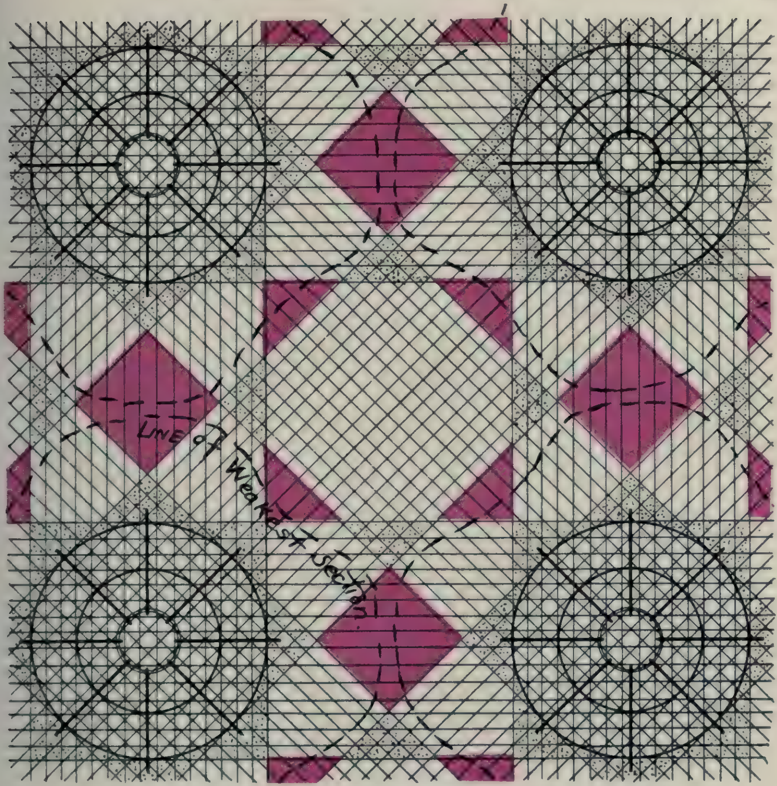


Fig. 9. (Mushroom System.)

thoroughly cured shows the development of cracks due to the stretching of the steel approximately along the dotted lines in the figure, noted as planes of greatest weakness. These planes or lines of greatest weakness pass through or close to the point of maximum moment on a line directly between the columns and to one side of the center of span in line diagonally between the coumns. Between these four lines of greatest weakness in the center of the panel we have an approximately circular flat plate.

To demonstrate these lines are nearly or approximately the points of maximum moment in the slab it is necessary that we consider the action of the compressive forces in the top of the slab through the central portion reinforced in two directions. Take for instance two planes parallel to the rods at right angles to each other through the center of the slab. We have the compressive forces along one line aa acting towards each other and on bb along another line acting towards each other and towards the center.

We know these forces can be held in equilibrium by forces directly in the line of bb and aa and also along the lines forming a square joining points b a b, etc.

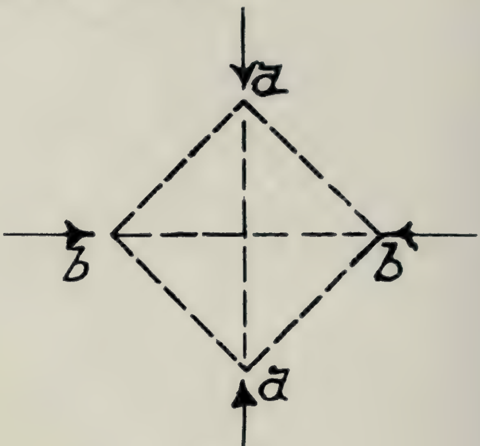


Fig. 10.

Evidently where the forces can be taken care of laterally they will not accumulate towards the center as they do in a simple beam and as the sum of the compressive forces equals the sum of the tensile forces the strain in the steel will be less than it would be with a one way reinforcement for its share of the load. In fact as Prof. Grashof and Dr. Eddy have shown in their flat plate theories the bending moment of a circular flat plate of homogeneous material (supported along its outer edge) at the center is zero and it seems quite probable that the compression in our concrete at the center will similarly be zero, though this may not be true of the steel since the strain developed in the steel at the outer part of this area where the moment is a maximum must be balanced by the stress in the steel along its length across the area at the bottom (except as relieved by the bond), constituting the principal difference between the relative distribution of stresses in the two types. In other words we have a curious cross or blending of a homogeneous flat plate and independent beams.

Now the *elastic limit of medium Open Hearth steel is a fixed quantity which varies little and we can readily determine

* The elastic limit here referred to is the limit of elasticity of shape as determined by a slowly applied load, as distinguished from rigid proportionality of elongation.

this value. We have only to test a panel of the finished construction, until the first yielding of the steel and we can then by simple proportion determine an exact working load for a known tension in the metal and our seemingly difficult problem has been solved in a simple and exact manner far more accurately than would be possible by attempting to determine Poisson's ratio of lateral extension under direct compression.

For the mushroom system in this manner we can readily determine that M_x equals $\frac{WL}{50}$ and for this system the author has adopted 13,000 lbs. as the working stress on the steel. This analysis gives a means for exact computation of strength in an extremely simple manner without bringing into consideration the knotty problem of web stress, since the web stresses are far less severe than in a beam due to the lateral arch or ring action by which the increments of flange stress are carried from one side of the slab around to the other.

For deflections this formula becomes

$$\Delta = \frac{1}{7000} \frac{WL^3}{\Sigma A_s d^2}$$

ARTICLE 3.

Accuracy and Applicability of the Above Formulae.

No formula based on the elastic relations of materials can be accepted as demonstrated unless its mate for deflection may also be depended upon. In other words an elastic theory in order to be accepted must demonstrate its accuracy by the elastic deportment of the material to which it is intended to apply.

It is interesting to note in connection with the similarity of the construction to the circular flat plate that the general formula for deflection may be reduced to approximately the general form for the deflection of a flat plate as given by Dr. Eddy in the year book of the Society of Engineers of the University of Minnesota, 1897-1901. This formula is as follows:

$$Z = -\frac{3(1-k^2)}{16\,Et^3} qr_1^4$$

in which Z is the deflection, q the unit load and r the radius and t the thickness. The other quantities may be treated as a coefficient and we have then the unit load times the fourth power of the radius divided by the cube of the depth but I the moment

of inertia which enters into this is $\frac{t^3}{12}$ equivalent to $A_S d^2$ for a unit width of reinforcement and spacing of bars.

Now in our WL^3 we have for a square panel the equivalent of qr^4 since in the denominator if a represents the moment of the steel per unit width the number of unit widths are multiplied by a portion of L since we make the mushroom head a fixed proportion of the size of the panel, hence the flat plate formula for deflection reduces to a form equivalent to the formula derived from consideration of relations of the reinforcement to beam formulae.

Coefficients Constant or Variable.—We have noted the general coefficient B and D and we have determined them for a specific type of construction. These coefficients can be used for any size of panel if they are constants and not variables. The values may be made constant by fixing the diameter of the mushroom head and width of belt to identically or approximately the proportional diameter used in the test panel from which the coefficient was determined. In other words if the diameter of the mushroom head and corresponding width of belt is kept within the limits of 7-16 to 1-2 the minimum distance between the columns the coefficient for deflection and bending moment is constant, otherwise it becomes an extremely variable quantity increasing in value several hundred per cent as the width of the belt is decreased to ¼L, hence coefficients of this character must be applied rigidly to proportional types of reinforcement until the law of their variation is accurately determined.

We have thus far determined the formula for this type of construction where the panels are square and we will now proceed to determine the coefficient where the panels are rectangular, that is instead of having one dimension L for the panel we have L_S for the short side and L_L for the long side.

The difference in distribution of the internal stress in the panel between the case of the square panel and the rectangular panel lies in the fact, that the line of weakest section in the central part or area of the panel is an ellipse instead of a circle and it would seem that for such a distribution the bending moment should be taken as increasing with the major diameter of the ellipse. In other words the external moment would be a coefficient times L_L; L_S only entering into the equation in deter-

Fig. 11. View of floor of Hamm Brewing Company's Stock House, showing tanks partly placed.

mining W and in determining also the diameter of the mushroom head which should be kept within the previous fixed limits.

Tests on a large number of panels in which Ls varies from six-tenths of ^{L}L to unity shows that this coefficient remains practically a constant taking the longer side of the panel as L in the general formula.

For example take the case of a panel of the Hamm Brewing Company's building, shown in figure (11). Panel 22′-10″ by 26′ 0″ loaded with four tanks 10′ in diameter, 15′ high full of water. The load as the writer figures it would be equivalent to 200 tons, uniformly distributed load. The floor slab was 14″ thick at the outer edge and pitched upwards 3″ to the center and reinforced with twenty-five ⅝″ rounds each way. Taking an equivalent depth of 15¼″ as the distance from center of steel to the top we have the following equation.

$$\triangle = \frac{1}{7000} \times \frac{(400)\ (26)^3}{(4 \times 25 \times .3)\ (15.25)^2} = .144''$$

Another panel in the same building, 20′ 10″ by 20′ 8″. Same loading thickness and reinforcement.

$$\triangle = \frac{1}{7000} \times \frac{(400)\ (20.83)^3}{(4 \times 25 \times .3)\ (15.25)^2} = \text{a full } 1\text{-}16''$$

These figured deflections proved exactly equal to the measured deflections as nearly as the engineer of the brewery could determine by marking the same with a knife edge.

We will take another case. Test of the State Factory building at Stillwater, Minn., Mr. C. H. Johnston, architect, shown in figure (12.) Size of panel 19′ 9″ by 20′ 8″. Thickness of slab 8″. Reinforcement seventeen ⅜″ rounds each way. Test load, 450 lbs. per foot over the full area.

$$\triangle = \frac{(180)\ (20\ 66)^3}{(7000\ \ 4 \times 17 \times .11)\ (7.25)^2} = .573'' = 9\text{-}16''$$

the reported deflection.

The Hoffman building, Milwaukee, figure (13). Test load 142 tons. Panel 17′ 0″ by 16′ 8″. Reinforcement seventeen ⅜″ rounds each way. Slab 8½″ (7½″ rough and 1″ finish.)

$$\triangle = \frac{(284)\ (17)^3}{(7000)\ (4 \times 17 \times .11)(7.8)^2} = .437 = 7\text{-}16''$$

the measured deflection.

Another example: Test of the John Deere Plow Company's buildng in Omaha. Panel 18′-9″ square. Reinforcement sixteen ⅜″ rounds diagonally and fourteen ⅜″ rounds directly from column to column. 7″ slab, in rough with strip fill added later about 2¼″ thick and a ⅞″ finish floor of maple. This we find an equivalent to a slab of about 8¾″ concrete as far as deflection is concerned, the strip being a 1-3½-4 mixture.

$$\triangle = \frac{(160)\,(18.75)^3}{(7000)\,(6.6)\,(8)^2} = .356'' \text{ or } \tfrac{3}{8}''$$

the deflection measured.

Test of 96 tons gave ½″ deflection in this case showing a little

Fig. 12.

yielding of the strip fill which had not had sufficient time to become hard and as rigid as it would have been with a longer period.

Figure (15), test load on the floor of the M. Born building. Panel 15′ 6″ by 20′ 0″. Slab 9½″ from the center of the steel to the top, 10¼″ or 10.25″ thick. Reinforcement, eighteen ½″ square bars twisted. Load 240,000 lbs.

$$\triangle = \frac{1}{7000} \times \frac{(240)\;(20)^3}{(18)\;(9.5)^2} = .168'' = .014'$$


Building for John Hoffmann & Sons Co.
FERRY & CLAS. ARCHITECTS.
NORTHWESTERN TILE COMPANY.
CONTRACTORS. MILWAUKEE. WISCONSIN.
TURNER MUSHROON SYSTEM
TEST LOAD ON SPAN 16'-8" x 17'-0"
LOAD 142 TONS OR 1000 lbs per □
DEFLECTION 14/32"

Fig. 13.

Fig. 14.

John Deere Plow Co. Building, Omaha. Test load 96 tons, panel 18' 9' square.

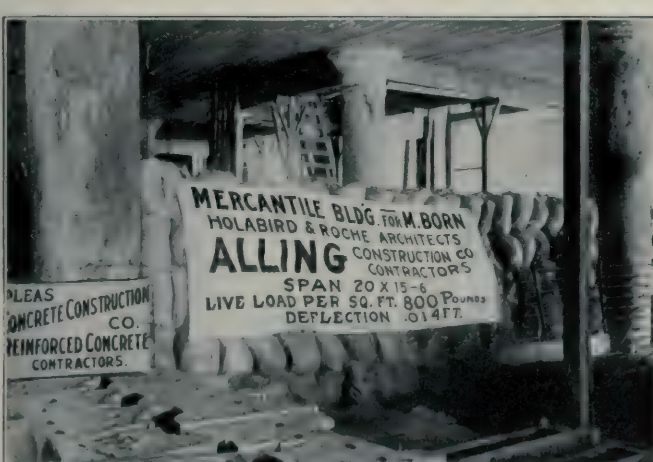


Fig. 15.
Test load on floor of the M. Born Building, Chicago.

Figure (16), view of the Moos building, Chicago. Panel 18′ 11″ by 20′ 0″. Slab 8″ thick. Test load 230,800 lbs. Reinforcement twenty-three ⅜″ rounds each way.

$$\triangle = \frac{1}{7000} \times \frac{(231)(20)^3}{(10.2)(7.25)^2} = .493''$$

the measured deflection $\frac{33}{64}''$

Fig. 16.

Test of Moos' Building, Chicago. S. M. Crowen, architect, Chicago.

Note—This test, as also that shown in Fig. 16b, was made before the concrete had been given time to harden sufficiently to deport itself in accord with formula based on older and harder concrete.

Excess deflection in tests made before concrete is thoroughly hardened.

Fig. 16b.

Test of second panel J. and B. Moos Building, Chicago. S. M. Crowen, architect.

Computed deflection:

$$\triangle = \frac{1}{7000} \times \frac{(240)\ (20)^3}{(10.2)\ (7.25)^2} = .511''$$

Measured deflection 11-16″ is 3-16″ greater than computed due to testing when concrete was *but five weeks old.*

The building department of Chicago criticised the result of this excellent test and did not appear to possess sufficient familiarity with the properties of concrete to recognize this as a remarkable showing in view of the short time which the concrete had had to harden.

EFFECT OF SPLICE ON DEFLECTION UNDER LOAD.

Test Load.

View from Street Front.

Bostwick-Braun Wholesale Hardware House, Toledo, Ohio.

Panel 17′ square. Reinforcement twenty 3/8″ round rods each way. Splice made in the center of the panel. Load applied, 1,250 lbs. per square foot in the center of the panel and 750 lbs. per square foot at the sides. Computed deflection as a monolithic panel:

$$\Delta = \frac{1}{7000} \times \frac{290 \text{ x } (17)^3}{(80 \text{ x } .11)\ (8.2)^2} = .346''$$

The measured deflection was 7-16″, or 3-32″ greater than the computed for a monolithic panel, which is attributed to the splicing.

Figure (18), view of the Merchants Ice and Cold Storage building. Panel 17′ 0″ by 18′ 0″. Test load 550 lbs. per square foot of brick, leaving 4′ square each corner unloaded.

$$\triangle = \frac{1}{7000} \times \frac{(133)\ (18)^3}{(7.5)\ (7.25)^2} = .28''$$

Fig. 18.

*The Official Testing of the Floors of the Grellet Collins Building, Mushroom System, by the City of Philadelphia.**

The Bureau of Building Inspection required that portions of the floors of the building (said portion to be selected by them) to be tested with a gradually applied load of twice the actual working load the floors were designed to carry, and that while the floors were being tested the deflection of the floors due to bending were to be carefully measured, as upon the amount of deflection in proportion to the amount of the load, the building department could judge the safety of the floor.

To obtain the results carefully and accurately, five iron rods were hung at various points from the underside of the first floor,

* Information furnished through courtesy of John G. Brown, contractor, for the building.

and upon these rods targets were secured. The floor was then cleared, the targets carefully adjusted by means of an engineer's level, and the loading begun. The loading was accomplished by placing boxes loaded with heavy coated enameled paper (the weight of boxes previously determined), over the entire area to be tested; when one layer of boxes was finished, the deflection of the floor was noted on each target, then another layer placed on top and the deflection noted, following out this plan until the loading was finished. To load the first floor required two and a half days, and the load was allowed to remain over night. The floor was then gradually unloaded and the recovery of the floor back toward its original position was carefully noted upon the removal of each layer of boxes.

The result of the test on the first floor was so remarkable that it was decided to test the seventh floor. This floor was designed to carry 150 pounds to the square foot, and was tested to 300 pounds to the square foot (to read the deflections, nine rods with targets were used), the deflection here was even much less than on the first floor. *The deflection was only 3-32 of an inch.* As a matter of general interest the seventh floor was tested to two and one-half times the designed load, with a very small increase in deflection.

In each test conducted the results were so pleasing to the building department that it was officially approved for use in the first-class buildings of Philadelphia.

The testing for the city was under the direction of Mr. George Warner, Concrete Engineer, Bureau of Building Inspection, City of Philadelphia.

The deflections read by Mr. James Haldeman, Assistant Engineer in charge of Lines and Grades (east half), Section Six, Market Street Subway.

The test of the first floor was witnessed by:

Mr. Edwin Clark, Chief of Bureau of Building Inspection, City of Philadelphia.

Mr. Manton Hibbs, Structural Engineer, Bureau of Building Inspection, City of Philadelphia.

Mr. Geo. S Webster, Chief of Bureau of Surveys of Philadelphia.

Mr. Geo. E. Datesman, Principal Assistant Engineer, Bureau of Surveys.

Mr. Henry H. Quimby, Assistant Engineer Bureau of Highways, Bridge Engineer, City of Philadelphia.

Mr. Howard Richards, Member of Wilson, Harris & Richards, Architects and Engineers.

Mr. Chas. M. Mills, Principal Assistant Engineer, Market Street Subway and Elevated R. R.

Mr. Richard Erskine, Architect, and Mr. Norman Hume, Inspector of Construction, representing Mr. Grellet Collins, owner of the building.

Mr. Edward F. Cobb, Sales Department, Vulcanite Portland Cement Co., which company supplied the cement for the reinforced concrete work.

1525 Fairmount Ave., Philadelphia, Pa.,
May 4th, 1903.

Mr John G Brown, Witherspoon Bldg., Philadelphia, Pa.

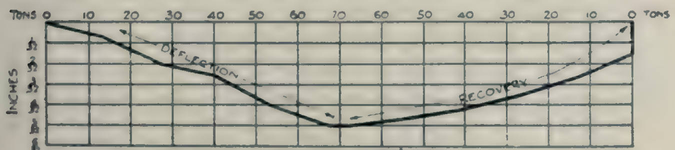
Dear Sir -Below please find detailed report of floor test on first floor of the Dill and Collins Building at the Northwest corner of Sixth and Cherry Streets, Philadelphia, Pa.

Load in lbs. per sq. ft.	Deflection in Inches Target Number 1	2	3	4	5	Remarks.
124	1/80	0	0	1/64	1/48	4-28-08 2.00 P. M.
200	1/48	0	0	1/64	1/24	
260	1/32-	1/32	1/48	1/48	1/16	4-28-08 5.30 P. M.
260	1/48	1/32	1/48	1/64	1/16	4-29-08 9.20 A M.
320	1/32	1/32	1/48	1/64	1/16	
375	1/32+	1/32	1/48	1/32	5/64	
435	1/24	1/24	1/48	1/32	3/32-	
500	1/24	1/16-	1/48	3/64	5/48	4-29-08 5.30 P. M.
500	1/16-	1/16	1/32	1/16	1/8	4-30-08 7.00 A. M.
560	1/16-	1/16	1/32	1/16	1/8+	
633	1/16	1/16+	1/16-	1/16+	5/32	4-30-08 11.40 A. M. (Final load)
633	1/16	1/16	1/16	1/16	5/32+	4-30-08 1.00 P. M.
500	1/16	1/16-	1/16-	1/16-	7/48	
375	1/16	1/16	3/64	1/16	1/8 +	4-30-08 5.00 P. M.
260	3/64	3/64	1/32	3/64	7/64	5-1-08 9.00 A. M.
124	1/32	1/24	1/24	1/24	1/12	
0	1/48	1/48	0	1/48	3/64-	5-1-08 2.00 P. M.
0	1/64	0	0	0	3/64-	5-1-08 5.20 P. M.
0	0	0	0	0	0	5-2-08 8.00 A. M.

I certify to report as being correct.

Jas. F. Haldeman
Asst. Engineer.

PLOT SHOWING DEFLECTION FOR EACH INCREMENT OF 6⅔ TONS LOADING
PLOT SHOWING RECOVERY ON REMOVAL OF EACH 6⅔ TONS LOAD



TOTAL LOAD 67⅔ TONS
633# AVERAGE UNIFORM LOAD PER SQ FT
AREA COVERED 214 SQ FT

The Testing of the Mushroom System Reinforced Concrete Floors.

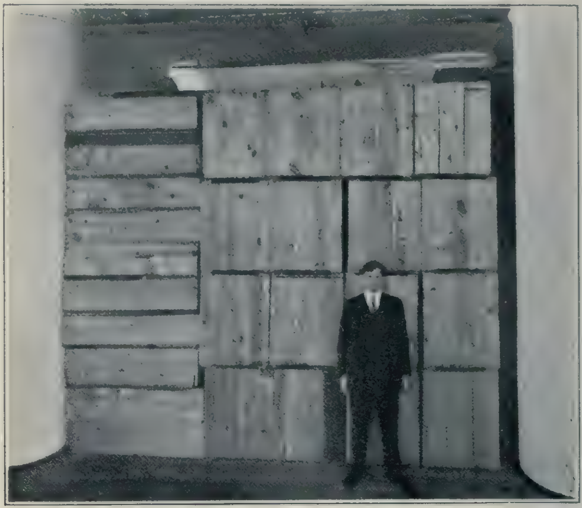
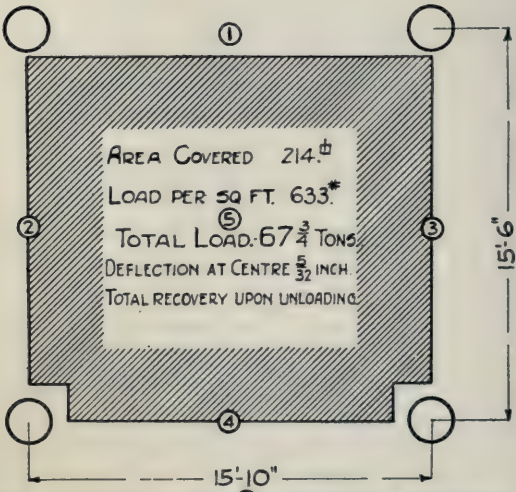


Fig. 19.

This floor designed to carry 300 lbs. to each sq. ft., or 30 1-10 tons. Now carrying 633 lbs. to sq. ft. or 67¾ tons.



FIGURES ENCLOSED THUS ⑤ REPRESENT POSITION OF TARGETS.

Fig. 20.

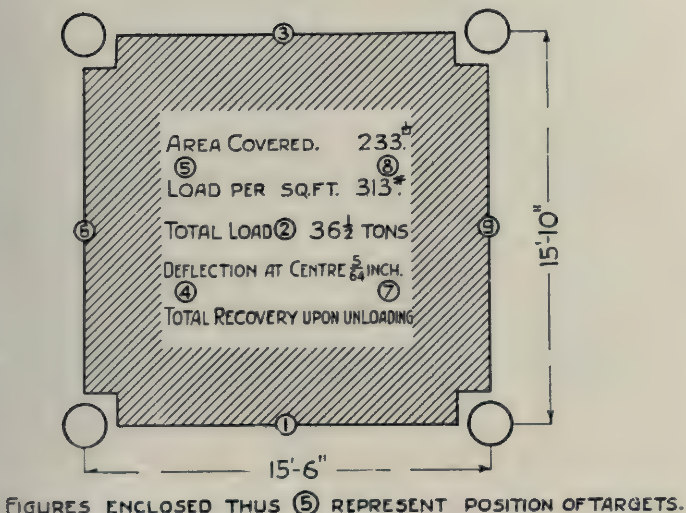
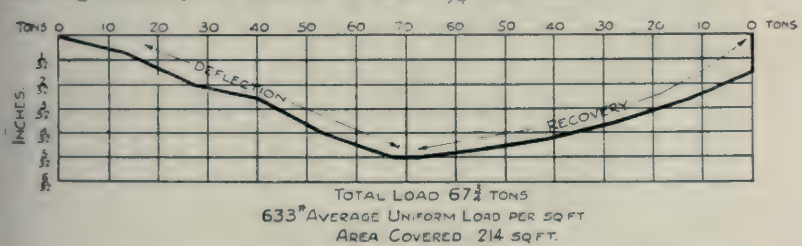


Fig. 21.

Plot showing deflection for each increment of 7¼ tons loading. Plot showing recovery on removal of each 7¼ tons load.



Test of Seventh floor construction of Grellet Collins building.

Computed deflection :

1st Fl. Slab 7¾″, strip fill 2″=effective depth, center of steel to top of slab 8″ 19 5-16 sq. yards each way.

$$\triangle = \frac{136 \times (15.833)^3}{7000 \times (7.5) \times (8)^2} = 5\text{-}32''$$

the measured deflection.

Fig. 22.

Testing of the Seventh floor.

Computed deflection reinforcement as before slab 7¾+2″ strip fill.

$$\triangle = \frac{73 \times (15.833)^3}{7000 \times 7.5 \times (8)^2} = .085'' = 5\text{-}64''$$

Fig. 23.

Showing Instrument and Rods with Targets for determining deflections. Underside of Seventh floor.

These examples are sufficient to give an indication of the practicability of figuring the deflections of almost any concrete construction with mathematical precision, provided that a good grade of concrete has been used and that this concrete has been given a reasonable time to cure and become fairly hard and rigid.

The majority of these tests were made with concrete from four to four and a half months old. The deflection would reduce somewhat had the test been made with concrete from a year to fifteen months old.

Thus it may be seen that while a multiple way system of reinforcement has the appearance of being exceedingly complicated from the standpoint of analysis, unlike multiple intersection steel frame work there is nothing easier to compute with certainty provided the engineer has a small amount of test data to work from which is applicable to the type with which he is dealing.

Owing to the small value of the coefficent W L developed under test for the moment it is evident that there is a very material economy in this type of design applicable not only to flat slab and column construction but also to the flat slabs in building work, square and rectangular slabs, supported by beams or walls.

ARTICLE 4.

Special Cases, Side Panels and Corner Panels.

We have yet to consider in the mushroom system two special cases.

(1) A side panel with continuous wall support on one side and two columns supportng the other.

(2) A corner panel having a wall supporting two sides and a column supporting one corner.

Test of a number of cases indicates that the same coefficient will apply since the deflection which would otherwise occur between the two columns on the sides which are replaced by the wall is eliminated by the wall and there appears to be no difference in the coefficient.

This fact is brought out best by the elastic deportment of the slab under load and the deflection formula. Take for example test of the Bovey building, figure (25). This test consisted of ninety tons on a panel 14′-10″ by 16′-6″ figuring center of bearing 6″ over the wall supports. Reinforcement seventeen ⅜″ rounds each way. Slab 8″ thick. Strip fill not hardened sufficiently to count.

Fig. 24.

Reinforcement of the First floor.

Fig. 25.

Load test of the Bovey building. 180 tons on two panels, 16x30 feet. C. A. P. Turner, Engineer, Minneapolis, Minn.

$$\Delta = \frac{(180)(16.5)^3}{(7000)(7.5)(7.25)^2} = .294'' = \frac{5}{16}''$$

the measured deflection.

Similarly for the mushroom panel in the corner of a building wth two sides resting on a wall, if the corner is filled out with the diagonal rods the same coefficients apply as determined by the test of the second panel in this same building.

An interesting feature of the deflection of a concrete panel of this type lies in the fact that the stresses seem to be very nearly self-contained within the panel. In other words the most careful measurement placing standards in adjacent panels, fails to reveal any negative deflection in the adjacent panels under loads of one and two hundred tons in the test of a single panel. Thus it is evident that there can be very little eccentric stress transmitted to the column in view of this interesting deportment of the slab.

Comparison of the figured deflections by this formula with the coefficient for simple beams shows that the slab is relatively considerably stiffer than beam construction, having a depth even double the thickness of the slab.

The advantage under concentrated loads in a multiple way system of this character lies in the fact that any concentrated load immediately brings into action the reinforcement of the entire panel and even more than that, hence under machinery such as printing presses which ordinarily set up considerable vibration in the timber or steel frame there is none preceptible with this type of concrete building.

ARTICLE V.

Deflection Limits.

Not a few of the city building departments have established the ridiculous requirement that the maximum deflection of a slab under test shall be the same percentage of the length of the span as that permitted or allowed for a deep beam. This proves that those responsible for the regulations were lacking in knowledge of the elementary relations of depth to unit strain and stiffness.

These relations are most easily deduced from the exact formulae for beams of a homogeneous material. Since the general relations of strength and stiffness would be similar in the case of

the beam of homogeneous material and the composite beam. Conclusions from formulae applicable to the former will apply to the latter. From the theory of flexure we have for strength

$$W = n \frac{S\,I}{l\,c}$$

where n=4, 8, 8 or 12, for the four different cases,* namely, simple and restrained beams loaded with W concentrated at center and W uniformly distributed. The requirement of stiffness, i. e., a given maximum deflection, limits the load by a different formula:

$$W = m \frac{E\,I\,\triangle}{l^3}$$

where m=48, 76.8, 192 or 384 for the same four cases.*

By equating these values of W the relation between $\triangle$ and S is obtained.

$$\triangle = \frac{n\,l^2\,S}{m\,c\,E}$$

This shows that the maximum deflection for the same unit stress varies as $\frac{l^2}{c}$ for beams of the same material, while coefficients n and m make a further wide variation. Such variations of course make it absurd to limit the permissible deflection to a fixed per cent of the span, as has been attempted by some ordinances.

Practical test under common conditions of partial restraint shows that in a span of forty times the thickness or depth of the slab or beam, i. e., l=40t (t=2c approximately) a deflection equal to $l \div 250$ will not materially injure the construction. Then by the above formula we may deduce that for l=10 t the maximum deflection should not exceed $\frac{l}{1000}$: In treating $\triangle$ as a per cent of the span, our unit of measure is the span. l. Hence the permissible deflection $\triangle$ as a per cent of l should vary directly as $l \div c$. or in other words directly as the ratio of span to thickness of slab or depth of beam.

As a practical constructor desires to test only to safe limits rather than anything approaching the ultimate limit he does not object to regulations twenty per cent more rigid than those noted provided the time of making the test is not less than four months after casting the concrete.

* In these formulas l=Span C=distance of outermost fibre from center of gravity; $\triangle$ =deflection S= maximum unit-stress, and I=the moment of inertia. m and n are coefficients.

ARTICLE 6.

Proper Time to Make Tests and the Significance of the Deflection of the Work Under Test.

There are many incorrect notions regarding the time necessary for concrete to harden. Under the most favorable conditions reinforced concrete work should be able to carry its rated load in ten days to two weeks. It should not be over-loaded at this period even when it has set up under the most favorable drying conditions. Some of the best Portland cements, high in silica, are very slow setting, requiring between three and four months to become sufficiently rigid and hard so that the deflecton of the work can be determined with mathematical precision.

When the work is tested before the concrete has thoroughly set the concrete squeezes together and the resulting deflection is abnormal. The steel may not have been strained severely and yet a large deflection has developed.

This can be quite largely removed by removing the load and promptly soaking the top of the slab with water which seems to expand the concrete somewhat and assist in its elastic recovery.

Such tests, however, have absolutely no scientific value and may do some permanent injury to the work.

Architects lacking in familiarity with the characteristics of Portland cement frequently specify that very severe tests shall be made at the end of thirty days. The experienced constructor, however, will never tie himself up to such a guarantee as the conditions affecting the hardening are far too variable to render computations by any known method of figuring of any value whatever in determining the elastic behavior of the material under severe load at this period. It is for this reason that the writer regards the majority of tests at the Technological Schools as of comparatively little value while those made in St. Louis under the auspices of the United States Geological survey on specimens which were hardened or cured under what might be termed hot house conditions and the time at which the specimens attained a fair degree of strength is no criterion whatever as to what the cement will do in practical use on the work.

In cold, chilly weather the writer has seen an entire floor of concrete remain dormant and at the end of two weeks be in almost as soft a condition as when the concrete was first placed. Ten days thereafter during a spell of warm weather the cement

hardened up in good shape and the centers could be removed. The writer was called about eight hundred miles by telegram in this instance, the assertion being made that the cement was worthless which would have proved, if correct, a very serious matter. The difficulty was that the superintendent had used water practically ice cold and the weather remaining chilly the cement had remained dormant during this entire period. This is one of the dangers in reinforced concrete construction, the failure to appreciate the fact it requires time for concrete to thoroughly harden, that the cement is active in hardening dependent quite largely on its temperature.

Concrete may be placed in the climate of Northern Minnesota in December, freeze up, thaw out in March and be as soft as the day it was placed, and after four weeks of proper treatment, wetting down and seeing that the temperature is near sixty, it may harden up in excellent shape.

In general a slab or beam of reinforced concrete is not in fit condition to test until it is hard and practically dry throughout.

Another condition which greatly affects the strength and deflection of reinforced concrete and which sometimes happens in building work, is the condition that an open floor becomes soaked with water after the concrete has fairly set up that this water on the floor and moisture in the concrete becomes frozen and later when another story or the roof has been put on and the heating plant put in the building this ice and frozen moisture in the concrete is suddenly thawed out. The concrete then is in somewhat the same condition as a water pipe which has frozen up. Sudden heating expands the ice and strains the iron very materially if it does not burst the pipe. So with our concrete, leaving the concrete temporarily in a condition of severe internal stress indicated under load by very excessive deflections.

The writer has been up against a proposition in a case of this kind in which the city building inspector took a notion that he desired to make a test with the concrete in this condition. It is needless to say that he did not get very far with his testing.

In drawing conclusions from tests to determine the coefficients of the general formulae the concrete should be at least four months old. A reasonable rule for testing work is the application of not exceeding one and one-half times the working load at the end of nine weeks and double the working load at the end of sixteen weeks time from the placing of the concrete.

ARTICLE 7.

Coefficients for Beams.

For simple beams supported at the ends we have

$$M_x = \frac{W\ L}{8}$$

Restrained and Continuous Beams.

Where we have a beam continuing over columns it becomes if reinforced over the columns a continuous or partially continuous beam.

For a uniform loading the beam to be truly continuous needs to be reinforced on the top flange only to the point of contraflexure. This condition of uniform live load we have only in the rarest cases and the conditions of a full load on one panel and none on the next is to be provided for.

With type III, figure three, in which the beams run in two directions and are monolithic with the slab and the slab is reinforced in two directions we have the slab rods running parallel with the beam capable of acting as tension reinforcement for top flange under negative bending and in this type of construction if the rods are properly placed and sufficiently lapped we have the condition of an ideal continuous reinforced concrete beam in which it is permissible to take D the negative bending moment over the support $\frac{W\ L}{12}$ and in the center of span $\frac{W\ L}{18}$ Where the beams frame into columns at the wall we make no difference in the bending. If however, the beam is merely supported $\frac{W\ L}{12}$ should be taken as the maximum positive bending moment.

For deflection the writer's tests on simple beams have been quite limited but we are in the habit of taking the coefficient (D) for the case of a beam supported at the ends, as approximately $\frac{1}{850}$ and for continuous beams which are cast with a heavy slab as in type III $\frac{1}{1000}$

These coefficients will of course vary somewhat with the type of reinforcement, degree of continuity secured, the character of the reinforcement in the slab and the thickness of the same and whether the beam has been cast at one operation or whether a splice has been made in the concrete. A splice in the con-

crete generally gives an additional deflection of one-sixteenth inch and even sometimes three-thirty-seconds inch in a beam of 20′ span when tested to double its working load.

A large amount of test data would be necessary to cover all types of beam reinforcement with anything like the degree of mathematical precision of the formula by which the deflection of a mushroom slab may be computed.

ARTICLE 8.

Division of the Load Over the Beams Where the Beams Run in Both Directions.

This is a question on which writers seem to be somewhat divided. In a square panel the direct division of the load would be along the diagonal lines of the panel, the area, if we should disregard the slab action in distributing the load being that supported by the triangles formed by the diagonals of a panel the base of which is one of the beams. If we disregard the action of the slab in distributing the load the bending moment on the beam would be $\frac{W\,L}{6}$ instead of $\frac{W\,L}{8}$ for a simple beam. The slab however, acts in a manner similar to the flat slab of the mushroom system transferring no small portion of the load directly to the column. Hence it appears more reasonable to take the load upon the beam as uniformly distributed and where the panel is rectangular in a similar manner to the treatment of the mushroom system and as would apparently be indicated by the consideration of the elliptical plate we would take the load as divided between the short and long beams on the sides of the panel in proportion to the length of the respectve sides or beams.

It may be stated that this is a radical departure from the views of all authors on this subject but it might also be stated that the formulae suggested by these authors are lacking in accuracy to the extent of several hundred per cent, and that further our tests of work designed in this manner have demonstrated to the author's satisfaction that this distribution gives the best practical results for variations in the sides of the panel of dimensions ranging from that of the shorter side equal to .6 of the longer side to unity. It is a fact, however, that this excellent type of construction is not amenable to the simple, certain and direct analysis of the flat slab mushroom type.

However by working out the appropriate formulae for the bending moment and deflection of the slab under this distribution of load if the coefficient (D) of the deflection formula proves a constant or practically so this outlined distribution of load can be shown correct and this seems to be the case from the test data at the writer's disposal which will be discussed later.

ARTICLE 9.

Square Slab Supported on Four Sides Reinforced.

Two Ways.—We note for a square panel that the external bending moment is less than that for a slab supported on two sides. If we draw the diagonals of the panel we have divided it into four triangles the equivalent of the load on each has to be transferred by the slab to the side support. Now the center of gravity of these triangles is one-third of the altitude distant from the support, in other words the center of gravity of the load carried to the support is one-sixth of the span distant from the support against one-fourth of the span distant from the support in the case of the slab supported on two sides.

Thus we have the external moment of the load for this condition two-thirds as great as in the case of the slab supported on two sides. In addition to this we have the reduction in the maximum moment due to the two way reinforcement or the flat plate action explained in our discussion applied to the mushroom system.

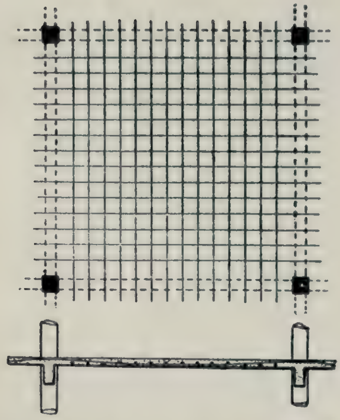
Owing to this flat plate action which gives us practically zero for the compression strain in the top of the concrete at the center and a maximum bending moment removed some distance from the center we cannot determine our coefficient B for bending more readily than by the same method of procedure adopted in the case of the mushroom system.

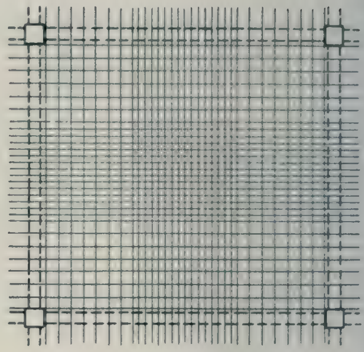
In this connection we must consider the various types of two way reinforcement across the slab.

(1) With the rods spaced uniformly across the panel both ways.

(2) With rods arranged with variable spacing as for example a belt across the center of the panel each way with rods spaced twice as closely in the central belt as they are in the outside belts.

(3) The condition in (1) and (2), and the condition of the





rods being kept close to the bottom of the slab over the beam support.

(4) The condition of the rods being raised towards the top of the slab over the beams.

The difference between the conditions (3) and (4) may be best explained by a concise statement of the deportment of a single panel under test.

ARTICLE 10.

Deflection of Square Slabs Under Test.

As we load a panel with the rods arranged as stated under (3) we seem to get, if the concrete has well hardened, the maximum deflection under the applied load.

Under condition (4), the deflection seems a little less than (3) at first, then gradually increases as the load is left on for twenty-four or thirty-six hours perhaps fifteen to twenty per cent as the concrete in the top porton of the adjacent panels yields under the negative bending. Such a test seems to show clearly that we cannot count on continuity over the support with a load on one panel with the adjacent ones unloaded. We may count somewhat on the arch action if the panel is sufficiently small and the slab thick as will be discussed later.

Returning now to a comparison of types one and two. Two requires about .8 as much metal in the reinforcement as arrangement one and seems to be a trifle stiffer as we would expect since the rods are spaced more closely at what seems to be the point of greatest bending.

The coefficient of WL for the external bending moment for one seems in the neighborhood of $\frac{W L}{30}$ and for two in the neighborhood of $\frac{W L}{36}$ while the deflection of coefficient (D) of type two appears to be $\frac{1}{10,000}$ from such test data as the writer has at hand.

This would be somewhat less if the rods were kept close to the bottom throughout though the writer prefers to keep them up slightly over the beam on the ground that as he views it this gives a somewhat better provision for shear, and a more satisfactory deportment of the slab under an extra severe test.

The deflection coefficient for type one the writer estimates $\frac{1}{10,000}$ with the rods raised at the support and $\frac{1}{10,500}$ approximately for rods near the bottom throughout.

ARTICLE 11.

Rectangular Slabs Reinforced in Two Ways.

Rectangular slabs reinforced in two directions the logical spacing of the reinforcement for the elliptic flat plate action is equal spaces in both directions over the belts or width (if a uniform spacing is not adopted) proportional to the length of the sides. The general formula for bending or deflection then remains the same except that L the span in the formula becomes a function of the length of the two respective sides which we will for convenience term "a" for the long side and "b" for the short side.

ARTICLE 12.

Computation of Strength.

Now if the load is distributed as recommended in the discussion of beams the load traveling to the long side of the rectangle would be $\frac{W\ a}{a + b}$

The load going to the short side would be $\frac{b\ W}{a + b}$ and the moments for these two respective parts of the total load would be these quantities, times their respective spans. In other words we would have

$$M_x = (B) \frac{W\ (a^2 + b^2)}{a + b}$$

$$M_1 = \frac{1}{12} \Sigma\ A_s\ .85d \times fs$$

(B) would be thirty and thirty-six respectively, for distribution of metal as under (1) and (2).

The deflection formula for this type of slab would be similar to the square slab except as above noted in which

$$\triangle = (D) \times \frac{W \left(\frac{a^2 + b^2}{a + b} \right)^3}{\sum A_s \quad d^2}$$

(D) should be the same coefficient in the respective cases (1) and (2) as derived for the square slab.

ARTICLE 13.

Computation of Deflection Applied to Practical Examples.

We will now proceed to apply this formula to the two cases.

First take the Minneapolis Paper Company's building, details shown in figure (26), photograph of test load in figure (27). Slab 7″ in the rough, 15′ 4″ by 21′ 6″ center to center of columns, strip fill 1¾″ and ⅞″ finished floor. Taking now the same proportion of the strip fill as effective as we usually do in the mushroom slabs, we find for load of 234,000 lbs.

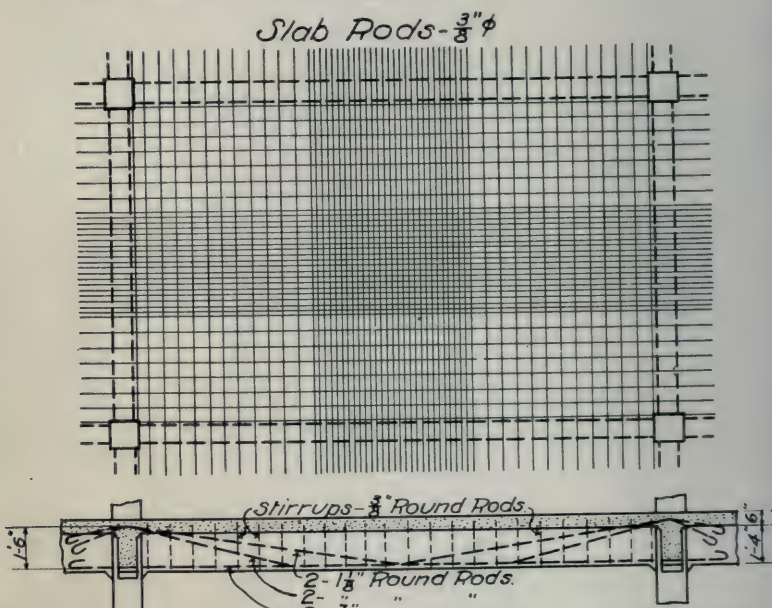


Fig. 26. Detail of slab reinforcement, Minneapolis Paper Co. building.

Fig. 27. Test of floor of Minneapolis Paper Co. building.

$$\triangle = \frac{1}{10,000} \times \frac{234 \times \left(\frac{21.5^2 + 15.3^2}{21.5 + 15.3}\right)^3}{(75. \times 11) \times (8)^2} = .30''$$

=deflection of slab at center as measured less the beam deflection.

Take for example the test made at the Smythe block at Wichita, Kans. Test load consisted of fifteen tons concentrated at the center over an area of 7 feet square equivalent to about 45,000 lbs. uniform load. Size of slab 20′ 9″ by 24′ 9″, 6½″ in the rough, 1¾″ strip fill. Reinforcement, ⅜″ rounds 5″ centers for the central third of the panel each way and 8″ centers outside third width each way. Then—

$$\triangle = \frac{1}{10,000} \times \frac{(45) \left(\frac{20.75^2 \times 24.75^2}{45.5}\right)^3}{9. \times 7.25^2} = .11'' \text{ or}$$

a full 3-32″ the deflection measured.

In the test at Wichita, the beam deflection, owing to the small load on the panel was practically negligible, and does not need to be considered in arriving at the true slab deflection.

Test of Minneapolis Knitting Co.'s building:

Slab 5½″ thick with 1¾″ strip fill panel 16′ 4″ × 15′ 8″ Reinforcement ⅜ o rods 4″ centers each way.

$$\triangle = \frac{1}{10500} \times \frac{160 \times \left(\frac{16.33^2 + 15.66^2}{32}\right)^3}{(10.4) \times (6)^2} = .167'' = 5\text{-}32'' +$$

The measured deflection agreed identically with that figured.

The writer's experience on this class of construction has not been as extensive as that connected with the mushroom system and the probable accuracy of the determination of these coefficients is in his judgment plus or minus four per cent while the probable error in the coefficients for the mushroom system is not greater in his judgment than plus or minus two per cent, these tests being made on concrete which is between three and three and one-half months old.

Fig. 28. Test of Minneapolis Knitting Co. building. 80 ton load.

ARTICLE 14.

Slabs Supported on Two Sides Reinforced in One Direction.

Such a slab may be considered in the light of a wide beam and figured accordingly. The deportment of a slab of this character under test is similar to that noted in slabs reinforced in two directions. That is for a single panel test, reinforcing bars which are simply carried up over the supports and with no continuous reinforcement over the top of the adjacent panels cannot be legitimately figured as continuous for live load though they may be figured as continuous for dead load only; on the basis of $M = \frac{WL}{8}$ for the live load moment and $\frac{WL}{24}$ for the dead load moment in the center of the span; and $\frac{WL}{12}$ over the support for the dead load.

Slabs Supported on Two Sides Having Two Way Reinforcement.

Such a slab would be in a measure represented by expanded metal reinforcement; in other words in which the tension members cross each other diagonally making an angle of twenty-two to thirty degrees, with a line normal to supports.

It is a well known fact that expanded metal seems to develop considerably more than the apparent cross sectional value of the material. As ordinarily made expanded metal is a rough, sheared, torn fabric, which cannot be expected to act as a tension member with the degree of uniformity of rods of uniform section, running in similar directions to the fabric members.

Such tests as the writer has made indicate a very marked advantage by such an arrangement of the metal. It is not however, generally advantageous to use this type because with a panel of moderate length the use of different length rods as we get to the sides are a disadvantage to its use though there are many cases where it can be adopted with a very material economy.

In the use of expanded metal sheets there is this disadvantage, that the sheets are made up to about ten feet in length and they are usually spliced by lapping one over the other, not a very satisfactory method, although combined with rods some very satisfactory work has been executed, using this method of reinforcement.

ARTICLE 15.

Short Span Slabs and Arch Action That May be Counted Upon in Their Use.

Heretofore we have discussed to some length long span slabs. A long span slab the writer will classify as a slab the ratio of whose thickness to length is so small that the possibility of its acting effectively as an arch is eliminated.

It is with long span slabs where the arch action is eliminated that formulas for bending apply with a high degree of precision. When, however, we come to test a short span slab which is part of a continuous floor there may be quite a large amount of arching in the slab carrying the load to the support without straining the steel to the extent that it would be stressed under the condition that there was no rigid skew back for it to thrust against. Where the span of the slab does not exceed ten times its thickness the writer would consider it permissible and good practice to figure the bending moment on the slab at $\frac{W L}{16}$ and would increase the moment, where the thickness is one-sixteenth of the span to $\frac{W L}{10}$ and for intermediate ratios of thickness to span intermediate values. Where span is more than sixteen thicknesses of slab it is to be figured as heretofore provided.

ARTICLE 16.

Value of Finish Coat, Strip Fill and Wood Floor from the Standpoint of Deflection.

Many engineers have an idea that a finish coat or strip fill or the like cannot act in connection with the slab to good purpose for the reason that the bond between the old concrete of the slab and that which is added at a later date will not be equal to the strength of the original concrete.

While this is true to some extent where the rough concrete is washed off and scrubbed with a steel brush, given a coat of neat cement grout before adding the finish coat or laying the strips and strip filling the concrete is nearly as efficient as though it were all cast at the same time, provided that the top coat or strip fill is given a reasonable time to set up hard before the test load is applied.

1-¾″ strip fill with strips 16″ on centers and a ⅞″ wood floor generally deflects the same as 1-⅛″ or 1-¼″ finish coat of concrete. The strip fill generally, however, if the strips used are 1-¾″ will somewhat exceed this nominal thickness since it is impractical to leave the top surface of the rough slab perfectly level and we can count as a rule that the actual thickness of the nominal 1-¾″ strip fill will not be less than 2-¼″ in the center of the slab though it may be a little less around the columns if the columns are poured in accordance with our standard practice.

If we assume that the bond between the finish coat and the old concrete is even thirty per cent of the strength of the original concrete we would still have an enormous factor of safety in view of the great area of the slab to take care of the horizontal shear between the two layers. This is a fact which is generally disregarded by those who are dealing with reinforced concrete. A slab the depth of which has been increased thirty per cent or such a matter by strip fill and the finish work is figured on the basis of the actual thickness of the rough slab only a suprisingly high degree of strength has apparently been developed by test on the basis of this rating while more conservative computation taking into consideration the actual value of this added thickness places the construction at its true worth.

In view of the enormous fire loss in this country going beyond the hundred million mark per year, attention to safe and economic features of non-burning types of building construction should be of interest to the general public for the reason that although they may insure their property and belongings and in this sense be protected from total fire loss they are paying in that insurance a direct tax sufficient to cover this entire fire loss together with profit to the company carrying the risk and the expense of placing the insurance and settling claims.

For strip fill, where strength becomes an object, we should figure on identically the same grade of concrete as in the slab instead of the weak, indifferent mud filling of cinders or natural cement or brown lime which is sometimes employed. Further by using Portland cement in the strip fill, the contractor will find that this filling hardens up and dries out much more promptly than any mixture of natural cement, brown lime, or Portland cement and lime enabling him to lay the finished floor at an

earlier date without danger of having the hard wood floor swell, buckle and raise up from the cleats to which it is nailed by reason of the moisture absorbed from the filling while drying out.

The writer has seen a case where lime was used with an idea of economy in a building completed in the fall and which the owners were in a hurry to occupy. This filling dried very slowly, seeming to have an affinity for moisture and when the finished floor was laid it caused it to swell so that it buckled up in places eighteen and twenty inches high, due to the swelling of the boards longitudinally, while laterally the swelling of this kiln dried maple was over fifteen inches in fifty feet. Six widths had to be taken out of the floor, the floor taken up and entirely relaid. This is the worst example of the kind which the writer has run across and it is well for the contractor to bear this point in mind.

ARTICLE 17.

Limitation of Working Stress on Concrete.

Thus far in our treatment of the subject of slabs and beams under bending, our attention has been directed, primarily to the determination of the strain on the metal rather than the equally important question as to the strain on the concrete. Evidently we cannot use with safety an amount of steel which would bring the working stress of the concrete beyond safe limits, such as eighteen to twenty per cent of its ultimate crushing strength with fair working stress upon the steel. We may most readily limit the compression on the concrete by limiting the percentage of steel to such a ratio that the working stress of the concrete will be within conservative limits. These ratios may be given as follows:

Simple beam, rectangular section one and one-half per cent of the area of the concrete.

For a T beam we may figure for this per cent using four thicknesses of the slab as effective area each side of the beam. Where however, the compression on the concrete on the bottom flange of a continuous beam at the support is excessive, due to more than the above limit of steel figured on a basis of the area of the beam not including the additional slab portion, additional reinforcement shall be used in the bottom flange for the necessary distance figured at 12,000 pounds per square inch in compression.

Percentage of steel for slabs reinforced one way shall be

limited in all cases to .7 per cent and for two way reinforcement one per cent counting rods both ways. These percentages should preferably be less and will generally be less if the work is designed in an economic manner. With an extra rich concrete, 1 : 1-½ :3, these percentages may be increased thirty per cent. However, in general the percentage of reinforcement in the slabs is about half of these maximum limits for the reason that a certain depth is necessary for stiffness and in usual practice in building construction the same rule governs in beams, i e., less steel than the maximum amount permissible.

ARTICLE 18.

Shear on Concrete.

It is not good policy to trust concrete in shear without a fair amount of reinforcement, especially at the supports. For slabs mix 1-2-4, twenty-five or thirty pounds per inch is permissible for concrete six or eight weeks old, provided that there is at least one-tenth inch of metal per lineal foot crossing the section. For concrete three months old fifty pounds per square inch is not out of the way. For footings forty pounds per inch is as high as should be used for a working stress in the writer's judgment for the reason that the concrete here is generally damp and filled with moisture and hence is not as rigid as where it is dried in a building.

It should be borne in mind that the concrete in setting shrinks and there is a tendency for the concrete to crack wherever there is a deficiency in the amount of reinforcement, hence a fair percentage of steel in the concrete, regardless of its exact position, tends to largely increase its shearing value by providing for shrinkage and temperature stresses.

CHAPTER IV.

Tests of Slabs and Floors.

ARTICLE 1.

Loads and Areas to Cover for Satisfying Tests.

In requiring that a building shall have a certain test capacity the question arises as to what is a reasonable test to make. In other words, what kind of a load is it necessary to apply to the various types of construction in order to demonstrate clearly whether they come up to the required standard or not. This question has considerable commercial as well as scientific importance.

In treating this subject we will need to refer again to the various general types of construction outlined in chapter II. With type I it is evident that the test of a single panel area between two joists means very little. The joist beams or girders would under a load covering the full area between them be strained ordinarily not exceeding one-third or one-fourth what it would be under the same loading per square foot covering four or five such panels with the joist in the center of the area. As to the main girders a single test of this character will be no indication whatever as to their strength.

With type II, a full panel loaded between columns the writer would regard as a fair test for the slab although the adjacent construction affords quite a little support in distributing the load over a somewhat greater expanse of slab than that which is actually covered by the load. Perhaps such a load where the columns are 16′ centers each way would be equivalent in effect to not more than 85 per cent of the load were the slab cut through on each side of the area loaded from main beam to main beam, while if we were to test the main beams fully it would require two full panel loads, one on each side of the main beam.

With type III test of a slab over the full panel area is a fair indication as far as the slab is concerned, for the reason that there can be, as we have pointed out, very little continuity action in the slab while regarding the beam if we are to make test of that it would be essential to handle the full amount of material recommended in case two.

For type IV, the full area between the columns is practically as severe a test as can be given the construction.

As the character and significance of the tests become more generally understood we may look forward to a gradual elimination of many inferior and weak types of construction, as when they are tested out rigidly under building ordinances which require the full equivalent of double the live load over reasonably large areas, weak details will become apparent and objectionable types of construction from the engineering standpoint will be gradually weeded out.

We have shown in the preceding chapters that natural concrete types of construction follow with a high degree of precision well defined laws of elastic deportment under test and these relations may be utilized in determining by the elastic deportment the characteristics of the construction from the standpoint of strength without going to the extreme expense of handling such enormous test loads as have been illustrated in the large number of cuts in the preceding chapter.

Thus if the half load applied to the construction gives a deflection consistent with that computed where the concrete has had time to properly harden, we may feel satisfied as to the results that may be secured under the heavier loading.

As noted in article 3, chapter II, regarding the web stresses in simple beams, this suggestion will not apply to that type of construction with equal force for the reason that the variation in the make-up of the beam will cause considerable differences in the deportment of the construction under test. Further, failure is not certain to occur where we figure the maximum moment in simple beams, hence the necessity for rigid testing of this class of construction.

ARTICLE 2.

Materials Used in Loading Slab or Beams for Test Purposes.

In selecting materials to be used for a slab test, if scientific information is desired regarding deflections, etc., the material should be such and so placed that there will be little, if any, arch action.

A very misleading type of test is shown in figure c, consisting of cast iron piled up in a manner which enables it to

Figure c. Illustrating a misleading test.

arch readily to a large extent from main beam to main beam. In this case the construction is practically type I with the joist girders five feet to six feet apart. This load is 1,500 pounds per foot. As far as the span on which it rests is concerned it was probably not equivalent to more than 1,000 pounds uniform load placed in a manner which would prevent arch action.

As far as the beam joist is concerned, having a slab of 6″ thickness, were the beam cut away, a fair amount of reinforcement should be able to distribute this load to the adjacent beams, assuming, of course, that the reinforcement was continuous. This being the case it is evident that instead of the load being carried by the one beam it is carried in part by the three, and the test viewed in this light reduces to a very moderate load on the construction, probably not more than eight-tenths its rated capacity.

A material such as gravel in bulk may arch somewhat perhaps to the extent of five to six per cent. With cement in sacks there may be also a small amount of arch action, but in view of the fact that the material is not rigid in form, as in the case of cast iron, this action can amount to very little unless special

pains are taken to lay the bags in a manner to secure such action, and even with the greatest pains it is doubtful if the bags can be placed on a large panel in a manner which would cause the arch action to amount to more than five or six per cent at the outside.

In general, we have called attention to the arch action which occurs in a number of slab and floor tests with the material used.

Figure d is a test by the Kahn people made at the St. Louis Exposition, using cast iron, in which there can be comparatively little doubt as to the distribution of the load.

Figure e is a test of a sample of their construction which, while quite imposing in appearance, would hardly be considered seriously by the thoughtful engineer.

The piling of material made in this test, as may be noted,

Figure d.

consists of cross piled pigs arranged in a manner to form a substantial skew back; then the rigid pigs are piled in between in a manner which combined with the height of the pile is conducive to most perfect arch action, sufficient to carry a large portion of the load to the supporting piers with little or no bending effect from this portion of the load on the construction beneath.

A test of this character is probably not considered in a serious light by those advertising it, although it hardly fails to

impress the less critical layman, and in that way may meet the commercial rather than the scientific requirements to the extent of satisfying the layman with whom the contractor may be dealing, by giving him an erroneous conception of the strength of the work.

In considering the degree or amount of arch action there may be in a pile of material we may note first that the arch action will be greater the greater the height of the pile as compared to its base.

Thus a pile of gravel 17′ square held in with a wall of sacks

Figure e.

filled with gravel on each side eight feet high might reduce the actual bending on the slab five or six per cent, as noted. If the pile were one-third of this height probably the arch action would not exceed from one-half to three-quarters of one per cent.

With a pile of pig iron carefully built the amount of arch action might readily become quite large, since the pigs are rigid, and if laid up carefully we could readily build a quite perfect Hindu arch which would carry over half the load to the support

or main beam without straining the portion which it is nominally the intent to test.

In general, the contractor naturally desires to use the material about the work which can conveniently be placed upon the panel or area to be tested and he should of course be allowed to do this, as the expense of making a reasonably conclusive test on a floor will frequently run into several hundred dollars.

Brick, cement in sacks, sand or gravel, stone, plaster and the like will frequently be used by the contractors if he has them at hand, instead of carting in pig iron from a distance, unless there is some object to be gained from a misleading test.

ARTICLE 3.

Care of Sacks.

The following hint is suggested for the benefit of the contractor who is inclined frequently to make tests by sacking up sand and gravel. Where the sand and gravel is wet, if it is allowed to remain in the sacks ten or twelve days the sacks as a rule rot and become worthless. As the present rebate is from seven and one-half to ten cents per sack, three or four hundred sacks destroyed in this manner through lack of knowledge, though a small item, is worthy of consideration.

ARTICLE 4.

Tricks of the Trade.

One of the tricks of the trade among those who care little for their reputation as reliable business men is to guarantee a floor for say a four hundred pound capacity, agree to make a test which will cover a panel insignificant in size and stress the beam on the side of the panel to less than one-third of the amount of the nominal test or in fact to an amount under test which is less by considerable than the rated working load uniformly distributed over the full floor area. This class of meaningless test is unfortunately too frequently accepted as conclusive proof as to the character and strength of the construction by the inexperienced architect.

ARTICLE 5.

General Problem of Multiple Way Reinforcement and Tests of Small Slabs.

In a building we have slabs which are partially continuous, reinforced in single and multiple directions, and the question as to the amount of the continuity effect in the comparison of the slabs becomes an interesting one, which may be best determined by the test of built slabs simply supported. A further interesting case in the problem of slabs from the mathematical standpoint is the discussion of slabs supported on two sides and reinforced in two ways.

We have noted under slabs supported on two sides that expanded metal develops greater strength than common theory would indicate for its cross sectional area under bending; that the material is a rough sheared and in a manner torn fabric; that the material could not reasonably be expected to develop the strength of rods of equal section running in directions similar or identical to that of the mesh; it has occurred to the writer that possibly since reasoning along the line of the flat plate theory that there is a certain amount of lateral arch action; that we might possibly separate out the individual rods of the two-way multiple system, treat them in pairs about their central intersection and get a line on the relative strength by comparing the components along the line of the rod which would result from the lateral distortion under direct compression parallel to the rod.

Reasoning on this basis, letting A (between the limits of twenty-five and forty-five degrees) equal the half angle between the rods let B equal the angle formed between a line normal to the supports and the rods; then analysis would give a general form for relative efficiency of the section a coefficient (taken as 4)—times sine A cosine A cosine B.

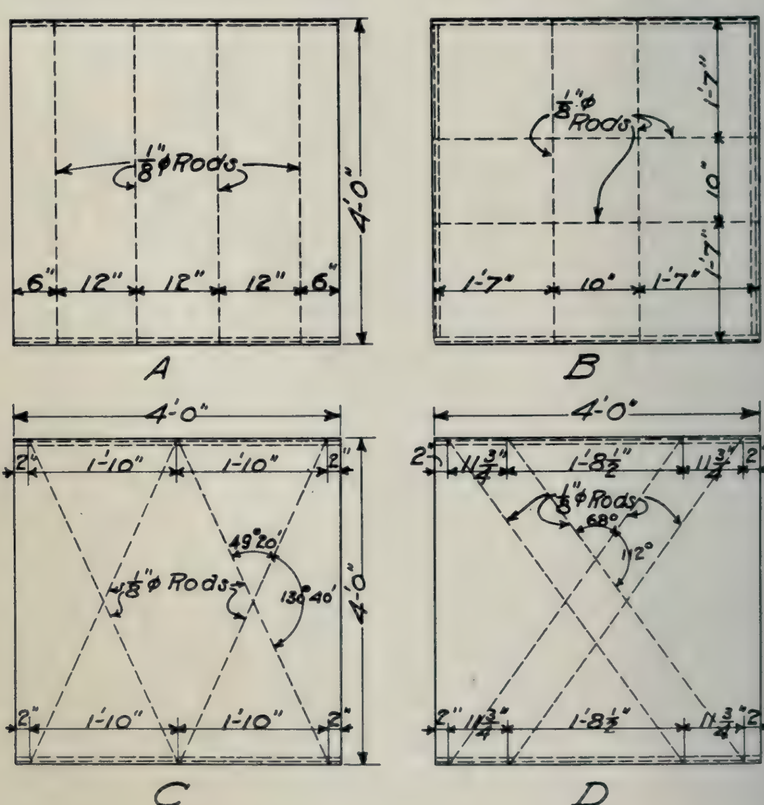
For the economy of metal four times sine A, cosine A cosine B^2 from which A equals thirty degrees is found to give the maximum efficiency of the metal. Where A equals B; thirty-five degrees for A gives the maximum efficiency of the cross sectional area without regard to the weight. The relative economy then becomes a maximum for A equals thirty degrees, equal to 1.25 the same weight of metal running directly from support to support in the case of a slab supported on two sides.

In the case of a slab supported on four sides angle B equals zero and the efficiency of the section then becomes four times sine A cosine A equals two times the efficiency of one way reinforcement from support to support for the same span, A being forty-five degrees as in the test of slab B.

For A equals B equals thirty degrees slab supported on two sides we have 1.49 for efficiency of section, which corresponds closely to such data as the writer has at hand regarding expanded metal.

Experimental data at hand is not sufficient to fully establish these relations, but they are suggested as an interesting probability.

Taking up now tests of small slabs, eight slabs were made as per figure. They were made four feet square, reinforced with No. 10 wire.



Slabs A, C and D were tested supported on two sides; slab B supported on four sides. It should be noted that the same number and size of rods were used in each case. All slabs were of uniform thickness, 2½″, and made with a concrete mortar in which no aggregate coarser than ¼″ was used.

Slabs B and D were loaded centrally on a block ten inches square; slabs A and C were loaded on a seven-inch wide timber placed across the center, so that in each case the load was a concentrated central load.

Comparing slabs A and B, the bending moment for the same load would be nearly identical in the two relative cases, thus differing from the case of a uniform load applied to similar slabs.

Results of these tests were as follows:

Slab	Failed Under
A-1	1,200 pounds
A-2	1,100 pounds
B-1	2,300 pounds
B-2	2,100 pounds
C-1	1,300 pounds
C-2	*
D-1	1,400 pounds
D-2	1,300 pounds

*Defective; rods placed too high for direct comparison.

The result of these tests indicates:

First, that pound for pound of metal, two-way reinforcement is doubly as effective as one-way reinforcement under concentrated loads in resisting a given bending moment.

Second, under uniform loads, two-way reinforcement is three times as effective as one-way reinforcement, since the bending moment is only 2-3 as great for a uniform load where the slab is supported on four sides.

Third, that diagonal reinforcement two ways with the same number of bars between thirty and forty degrees is considerably more effective than one-way reinforcement.

Fourth, that the tests on the diagonal reinforcement are not sufficient in number to prove an exact relation, nor can the test under concentrated loads be expected to give the same results as the test under uniform load for the reason that the combined shear and moment would tend to cause the concrete and possibly

the steel to fail at an earlier stage relatively than where the load applied is uniform.

A study of this character, simple though it is, will upset nearly all preconceived notions of the average structural engineer who would figure case C and D as materially weaker than slab A, while slab B would be considered, under concentrated load under the ordinary theory, as no greater strength than slab A, whereas it is practically double, and as pointed out, would be three times as strong under a uniform instead of a concentrated load.

These tests, it should be observed, are tests really of the strength of the steel in the combination, since the amount of steel used in this reinforcement was intended to be less in point of strength than the concrete.

The efficiency of the concrete where the steel is increased in cases A and B would be materially different from the efficiency of the steel used, since, where the steel increases the efficiency of the concrete in B strained in two directions would be perhaps fifty per cent greater than A strained in one direction, while the steel in case B is doubly as efficient as in case A under concentrated load, and three times as efficient under uniform load, pound for pound.

CHAPTER V.

Discussion of Theories and Elastic Properties.

ARTICLE 1.

Common Theoretical Errors and Their Orign.

A treatise dealing with concrete steel construction, owing to the present state of the literature or the art (authentic literature on the subject written by contractors being hitherto, nearly if not quite lacking), would be incomplete without a pointed reference to the gross errors in theoretical treatment common in works on this subject.

Referring now to "Reinforced Concrete" by Marsh, (Part V), he frankly states that unfortunately it cannot be stated that we have a thorough knowledge of the various properties of reinforced concrete.

"When properly combined with metal, concrete appears to gain properties which do not exist in the material when by itself, and although much has been done by the various experimenters in recent years to increase our knowledge on the subject of the elastic behavior of reinforced concrete, we are still very far from having a true perception of the characteristics of the composite material.

"It may be that we are wrong from the commencement in attempting to treat it after the manner of structural iron work, and that although the proper allowance for the elastic properties of the dual material is an advancement on the empirical formulae at first employed and used by many contractors at the present time, yet we may be entirely wrong in our method of treatment.

"The molecular theory, i. e., the prevention of molecular deformation by supplying resistances of the reverse kind to the stresses on small particles, may prove to be the true method of treatment for a composite material such as concrete and metal. This theory is the basis of the Cottancin construction which certainly produces good results and very light structures, and M. Considere's latest researches on the subject of hooped concrete are somewhat on these lines.

"Where, however, as is the case in most of the systems at present employed, comparatively large sections of metal are used for the purpose of directly resisting the stresses, we must treat the subject on the usual formulae based on the direct elastic resistance of the materials and the deformations which are produced in its various applications."

The writer thoroughly agrees with Mr. Marsh in this statement and it is with no small degree of surprise that we find later in his work that he recommends the Dunn formula for slabs reinforced in two directions. This formula is derived by disregarding the general equations of stress in an elastic solid which we are forced to employ to arrive at a logical mathematical de-

termination of the stresses in a solid such as a flat plate supported on four sides under bending.

The Dunn formula is based on the assumption that strips across a panel may be logically treated as independent beams and their deflections as strips (independent of the lateral support of the mass of which they are a part) may be compared in determining the distribution of the load to the side supports, without in any wise checking it by observation of the elastic behavior of the plate as a whole under test.

He then proceeds to criticise the semi-empirical formulæ of Hennibique, one of the most practical pioneer constructors in the field of concrete steel construction on the ground that the logic of the derivation of Hennibique formula is not clear although it has been tested and found practicably in accord with the results of a broad experience in actual work.

Pioneer work in any field is liable to be somewhat crude and while not a practical constructor Mr. Marsh must be credited with excellent work as a pioneer writer on the subject of reinforced concrete and it would be strange indeed should we not find a few algebraic monstrosities in the first notable attempt to get out a comprehensive treatise relative to this comparatively new industry. This would hardly apply to the later treatises in the field and the fact that formulae over three hundred per cent in error should be adopted by one writer after another can only be accounted for by the modern fashion too much in vogue of producing a new technical treatise by *copying* without verification the bulk of the matter from earlier writings on the subject.

As in steel construction we may force the load to a high degree to travel a predetermined path by arbitrarily placing the metal along that path so we may transmit the load to the support in reinforced concrete in an arbitrary manner across the panel supported on four sides by distributing the metal through the panel in an arbitrary manner.

Such distribution along the lines of Mr Dunn's formulae as advocated by Mr. Marsh would not give a fairly uniform coefficient for bending for different size panels and the writer would regard the distribution of metal as ill-calculated to secure the economic effect by elliptic flat plate action. As to results, the error from the standpoint of computed strength would be something like three hundred per cent. From the fact that Mr.

Marsh was familiar with the practical formula of Hennibique this theoretical error is even more surprising and although the logic of the rule adopted by Hennibique may not appear clear the fact that it was based on practical experience should give any theoretical writer not conversant with construction cause to pause and investigate before recommending one that is so grossly in error.

On page 356 of "Reinforced Concrete" by Marsh, edition of 1905, the formulae of Hennibique are given, for a rectangular slab reinforced in two ways.

ARTICLE 2.

Hennibique Method of Calculation for the Bending Moment of a Slab Reinforced in two Ways.

"The bending moment on built-in slabs is arrived at by a peculiar method. If w is the weight of the piece itself per lineal unit, and w1 the uniformly distributed load per lineal unit, the bending moment for a built-in beam is taken as

$$\frac{(w + w_1)\ L^2}{10}$$

a very customary allowance. The slabs being supported on all edges, the bending moments are considered in both directions. If L is the longer span and B the shorter span, there are supposed to be two bending moments—

$$M_L = \frac{(w + w_1)\ L^2}{10}, \text{ and } \quad M_B = \frac{(w + w_1)\ B^2}{10}$$

and the maximum on the slab is considered as the mean of these two, or—

$$M = \frac{M_L + M_B}{2}$$

The disposition of the reinforcements, crossing on another and being continuous over the supporting beams, is assumed to considerably diminish this bending moment. It is therefore reduced to one-third and the value for use in the calculations is taken as $\frac{M}{3}$. This value for the bending moment only applies to the continuous slabs."

It will be noted that this does not vary greatly from the coefficient adopted by the writer.

In this connection a few general remarks regarding responsibility of the theorist and the engineer contractor as regards the work are in order.

To the critics of his theories Hennibique is credited with this cutting and sarcastic remark, that he was not in the business of peddling formulae but of selling construction guaranteed to carry, without injury, a specified test load and that he assumed the financial responsibility therefore.

Unfortunately there seems no legal or commercial responsibility attached to the selling by publication of theoretical formula further than it shall appear as such in point of algebraic get-up. Whether it requires a construction three times as strong as necessary where a given guaranteed test load is demanded and places thereby the contractor that uses it at a disadvantage in commercial competition, causing him to lose the work that he has expended considerable money in figuring upon, or whether it results in a construction dangerous and unsafe and for which the contractor adopting it is placed in a position where he is compelled if financially responsible, to pay the bill is alike immaterial to the theorist publishing a formula grossly in error since there is no legal redress or opportunity for the victim to recover by suit. In other words the engineer contractor takes his own chances in applying any information regarded as authoritative if he uses it without careful investigation in his everyday business.

ARTICLE 3.

Character and Significance of Tests of Elastic Properties of Concrete.

In the usual treatise on concrete much stress is placed on the determination of the value of the modulus of elasticity of concrete. Now, bearing in mind the correct definition of concrete as an artificial conglomerate stone, we may legitimately question how much of the modulus determined on a diminutive sample belongs to the conglomerate aggregate, and how much to concrete proper.

On this question the writer's views may be extremely pessimistic. He would hesitate to stake a dollar of his hard earned money on computed elastic deportment based on tests of this character and think nothing of giving a fifty thousand dollar

bond guaranteeing the elastic deportment of work figured from coefficients determined by careful measurement of deflection, of business like samples of concrete sixteen feet to twenty-five feet square in the form of slabs under test, since he finds the greatest uniformity in his experiments conducted thousands of miles apart with work executed under the same general specification by many different gangs.

The writer is also equally pessimistic as to the propriety of the use of the modulus of elasticity in working out column formula since the use of such formula leads to dangerous details of construction, which in view of the apparent simplicity of the formula, is theoretically permissible.

Thus these formulae are derived as follows:

$P = f(A_c + e A_s)$ compressive strength of section;

$f =$ crushing strength of concrete;

$A_c =$ area of section of concrete;

$A_s =$ area of section of steel;

$e = \frac{E_s}{E_c}$ where $E_s =$ modulus of elasticity of the steel and

$E_c =$ modulus of elasticity of the concrete.

Now E_c varies greatly with age of the concrete. The concrete in setting shrinks and this shrinkage stress is totally disregarded in this formula.

Finally the conclusion from this formula would be that one reinforcing rod in the center of a column is just as efficient as any other distribution of longitudinal reinforcement and is much the more convenient in construction.

Since a more dangerous error from the standpoint of safety in erection could not well be made, the practical constructor on reading such literature as this, only wonders that the failures of concrete construction are indeed so few in number.

We have noted in an earlier chapter the tendency of the designer to imitate older types or forms of construction. This imitation of older methods in concrete steel construction has been carried to the greatest extremes by the "would-be" scientific theorist. With the older materials of construction, such as timber and structural steel, we base our formulae for deflection upon the modulus of elasticity of the material used, and also our

formulae for determining the distribution of stress through statically indeterminate combinations or frames.

In concrete construction, however, this modulus of elasticity determined in the usual manner does not apply to the natural types of reinforced concrete construction, because the strain in the concrete is not a simple strain in one direction, but a very complex distribution of stress in a number of directions. It seems to the writer that some of our theoretical friends have made a labored effort to apply a coefficient totally inapplicable to this type of construction, with the natural result that their mathematical work leads to no definite or valuable conclusions, but merely serves to cloud the perceptive faculties of those interested in the development of the business.

Again, even were it possible to draw valuable conclusions from the modulus of elasticity in compression, the question naturally arises as to what is the age of the specimen at which it is permissible to determine this constant.

Table cited shows the modulus of elasticity of concrete as determined by tests at Watertown Arsenal in 1899:

Mixture 1:2:4.

Brand of Cement.	Modulus of elasticity of loads between 100 and 600 lbs. sq. in.			
	7 days.	1 month.	3 months.	6 months.
Atlas	2,778,000	3,125,000	4,167,000	3,125,000
Alpha	....	2,083,000	4,167,000	3,135,000
Germania	2,500,000	....	3,571,000	4,167,000
Alsen	2,500,000	2,778,000	2,778,000	4,167,000
Average	2,592,000	2,662,000	3,670,000	3,646,000

From the table we note that the modulus for Alsen cement increases from 2,778,000 to 4,167,000 between the age of three months and six months; that the Atlas brand decreased from 4,167,000 at the end of three months to 3,125,000 at the end of six months; that the seven-day tests and the one-month-old tests are very much higher than could be depended upon in ordinary practice; that, although the six months tests of the Atlas and Alpha cements show retrogression under that of three months, if the specimens had been kept longer these values would

undoubtedly be increased to considerably greater than the maximum at the end of the six months period.

These being fair average values, upon what line of logic do some authors conclude that the ratio of the modulus of elasticity of steel to the above concrete is in the neighborhood of one to twelve or one to fifteen? Further, by what method of accurate mathematical analysis do they proceed to determine the exact amount of steel that it is permissible to use in a column and to delineate the entire makeup of the column by such extremely accurate ratios, if we consider them based on practical tests? Does it not look as though where the premise differs at least 100 per cent from the average values for ordinary working stresses that there might be a far greater difference between fact and the conclusions that may be arrived at by working from such an erroneous hypothesis?

The writer, at least for any work which he is called upon to guarantee, desires a method of figuring based upon a premise that is a little less doubtful and that does not involve the extreme variations embodied in such assumptions regarding the value of the modulus of elasticity. He prefers to *know* with a reasonable degree of certainty what his work will carry.

Much of the test data which has been worked up at the technological schools is of considerable value in a qualitative manner, where it is or may be nearly worthless from the quantitative standpoint.

Tests made on simple beams when the specimens have not sufficient age may be of considerable value as an indication of general deportment, though of very little value from the standpoint of quantitative results.

A series of tests in which large variations are noticeable in the value of specimens mixed in the same proportions and with the same materials, is not conducive to confidence in the character of the work or the uniformity of the conditions under which the specimens were cured. With this confidence shaken the quantitative values of the results are in question.

Experiments which are based on impractical forms of construction may be of value from the negative rather than the positive standpoint. They may indicate clearly what ought not to be done rather than what ought to be done in the field of practical design.

The writer is inclined to regard many of the tests made at Champaign, Ill., on columns, somewhat in this light, although expressing his full appreciation of their value. In other words, the tests at Champaign are of positive value in indicating what should not be done in column design. They indicate clearly in the writer's judgment, that hooping without the proper proportion of vertical reinforcement combined therewith, is not a suitable design for a column. At the same time, they give no indication whatever as to the value of a proper and suitable combination.

As to the tests on simple beams, the writer's opinion is that in the near future this information will be more interesting from the academic rather than from the practical standpoint of the concrete constructor, who will rarely, if ever, employ such a beam in his everyday work where it can be avoided.

Prof. Turneaure's conclusions resulting from tests made by him to determine the point at which incipient hair cracks develop in concrete under bending, are, as the writer understands them, that these cracks commence to develop as soon as the steel is strained to 5,000 or 6,000 pounds per inch.

A point that should be noticed, however, with reference to the cracks referred to by Prof. Turneaure (see page 39, Concrete Construction by Turneaure and Maurer), is the manner in which the beams were cured. "In some experiments made at the University of Wisconsin, 1901-2, a very delicate method of detecting incipient cracks was accidentally discovered. It was found that beams cured in water which were only partially dried before testing would, when tested, show very fine hair cracks at an early stage, and moreover, by watching closely, it was observed that preceding the appearance of a crack there would appear a dark wet line across the beam. Such a line would soon be followed by a fine crack. A larger series of tests were undertaken in the following year by a different set of experimenters who observed the same phenomenon."

In this connection the following facts should be observed, which do not appear to have been noted by these investigators:

First, that the curing of a concrete beam in water, then removing and allowing it to partially dry would leave the concrete in a state of internal strain. In other words, the beam would not be in a normal condition and conclusions drawn from it

would not be applicable to a similar beam cured in the ordinary way.

When we compare a simple beam with a continuous beam in which the main line of reinforcement is arranged so that it can carry shear as well as bending as it approaches the top of the beam at the support, we have very different conditions indeed.

First, comparing the stiffness of the simple beam to the stiffness of the continuous beam, we have the stiffness of one equaling five times that of the other, and the deformation of the concrete has become correspondingly small for the same maximum strain in the steel. Hence, granting that with an inferior design of simple beam, incipient cracks may develop at as low a stress on the steel as 10,000 to 12,000 pounds per square inch, we should not look for them in a properly designed continuous beam under 25,000 to 30,000 pounds stress per inch upon the steel.

While the writer's experience in practical construction causes him to differ from the position taken by Turneaure regarding the point at which incipient cracks are developed in concrete steel beams in construction generally, it is a fact that the material, when thoroughly cured, will stand only a certain amount of deformation in bending without checking; that this amount follows closely the limits suggested under the heading of permissible deflections; that in view of the limitation of concrete in this respect the writer is of the opinion that medium steel, standard specifications, will stretch about as far within its elastic limit as concrete can be reasonably expected to follow without checks or injurious effect upon the material.

Admitting this to be a fact there would be then no particular advantage in the adoption of high carbon steel, nor would it seem legitimate to allow a higher working stress than about 16,000 pounds per inch on the metal. The computed stress on the steel, however, should be made upon a rational hypothesis and to properly take into consideration the arrangement and character of the distribution of the steel through the work.

ARTICLE 4.

Work of United States Geological Survey.

In the experimental work conducted by the United States Geological Survey, under an advisory board, no member of

which had been commercially engaged in the art of reinforced concrete construction, some rather amusing results have been shown. For example, in comparing tests of slabs, reinforced in one way and supported on two sides, with slabs reinforced in two ways and supported on four sides, they apparently conclude that the Dunn formula is not far out of the way. We may note the following facts:

That the bending moment of the load on a slab reinforced in two directions would be but two-thirds that of a slab reinforced in one direction; that the steel in the two-way reinforced slab tested was double that in the slab reinforced in one direction; that the failure took place in the concrete and not in the steel. Hence, the tests were of value only in so far as they gave a line on the capacity of the concrete as strained by the two-way reinforcement, but gave no indication regarding the value of the steel placed in this combination. In other words, the value of the concrete when strained in two directions will be capable of developing a resistance or strength to resist a bending moment about 50 per cent greater than where reinforced in one direction only. But it by no means follows that it requires more than one-half the amount of steel arranged in two directions to develop this capacity.*

These are questions which would have been accurately investigated had the advisory board, known as the Joint Committee on Concrete and Reinforced Concrete, possessed that degree of practical knowledge of the subject under investigation which seems to be acquired only by continued work in the commercial field of actual construction. So far as those engaged in the industry can see, no tangible results of value to the real workers in this field can be shown to justify the expenditure of the $100,000 government appropriation for this purpose and for which this committee has assumed the responsibility.

ARTICLE 5.

Shrinkage Grip of Concrete on Steel.

The high value of the shrinkage grip sometimes called adhesion of concrete to steel rods imbedded therein, has been long known and depended upon by nearly all of the early pioneers in the field of reinforced concrete on the continent. Numerous

*See page 74, Tests of Slabs, etc.

tests have been made to determine the adhesion of concrete to plain rods of different forms, with results varying from 200 to 750 pounds per square inch of the surface contact.

The following table is quoted from Concrete Construction by Turneaure and Maurer:

Authority.	Kind of Concrete	Steel Rods. Kind.	Steel Rods. Size inches.	Depth Embedded, inches.	Adhesive strength lbs. in
Feret: Ciment Arme, p. 755	1:2:4	Plain round	0.8	2¾	237
	1:2:5	Plain round	0.8	2¾	190
	1:3:4-½	Plain round	0.8	2¾	237
	1:3:6	Plain round	0.8	2¾	195
Hatt: Proc. Am. Soc. Test. Mat., 1902	1:2:4	Plain round	⅝	6	756
	1:2:4	Plain round	7-16	6	636
Emerson: Eng. News, Vol. L1 1904, p. 222	1:3	Plain round	½	6	512
	1:3	Plain flat	¼ x 1	6	293
	1:2:4	Plain square	1 x 1	10	587
	1:3:6	Plain square	1 x1	10	478
Talbot: Bull. No. 8, Univ. of Ill., 1906	1:2:4	Plain round	½ & ⅝	6	438
	1:2:4	Plain round	½ & ⅝	12	409
	1:3:5-½	Plain round	½ & ⅝	6	364
	1:3:5-½	Plain round	½ & ⅝	12	388
	1:3:5-½	Cold rolled shafting	1 & ½	6	146
	1:3:5-½	Mild steel flat	3-16x1½	6	125
	1:3:6	Tool steel round	¾	6	147
Withey: Bull. Univ. of Wis., 1907	1:2:4	Plain round		6	401
	1:2:4	Plain round	9-16	6	504

We note on the experiments of Feret that the adhesive strength is very low, about one-third that shown by Hatt and one-half that shown by Emerson. The difference would seem to be most readily accounted for on the ground of the different amount of moisture in the concrete used by the several investigators.

A wet concrete gives a grip far in excess of a dry tamped concrete, hence the importance of using a wet mix in practical construction.

It should also be noted that the mixture used where the smaller values are obtained, with the exception of two by Feret, was a rather weak mixture. The low values found by Feret

would seem readily accounted for by the prevalent French custom of tamping concrete rather than pouring it, as is done by the Advanced American constructor. This difference in treatment of the concrete may be readily used to vindicate the large advantage that may be obtained by deformed bars.

Admittedly they will show up much better than the ordinary plain bars in this class of concrete, dry tamped, which it is not desirable or permissible from the practical standpoint to use in the work.

The old fashioned notion was that good concrete could only be made by a dry mix and tamping. Combined with this class of concrete the deformed bar was a decided improvement.

At present, however, the advocates of dry tamped work are either all dead or converted to modern methods, and the advantage in the use of the deformed bar in ordinary work has died with them.

Adhesion to the flat bars seems inferior and they should preferably be avoided in the work. A round rod is most easily surrounded in pouring and is the ideal form of rod to employ.

ARTICLE 6.

Question of Fatigue.

Steel when strained up to one-third or a little beyond one-third of its ultimate strength deteriorates in quality and becomes brittle and finally is liable to break without apparent cause. Where the stress is of two kinds its destruction is much more rapid, due, as the writer looks at it, to the fact that the molecular change in structure due to tension and compression are of the reverse order; leading to a lack of homogenity in the structure and the rapid wearing out of the piece.

In concrete it seems that the same general law holds true. Repeated stress exceeding more than about a third of its ultimate strength is liable to deteriorate the material, but within this limit, as with steel, there is probably little if any damage done by repetition of strain.

This conclusion as regards concrete is based on a number of observations of the material in the form of slabs, subject to the repeated and continued jar of machinery under the most try-

ing conditions for a number of years. Apparently the requisite for this material to properly withstand repeated vibratory stress without injury is a first class, rich concrete, properly mixed and thoroughly cured.

CHAPTER VI.

Columns and Column Formulae.

ARTICLE 1.

Types of Columns.

The following may be briefly stated as a requirement for suitable design for reinforced concrete columns in building construction.

First, that the longitudinal reinforcing metal should be toward the outer portion of the column in order to properly provide for any tendency to bend or deflect.

Second, that the bars should be banded or tied together to maintain them in the desired position.

Third, that the bands or ties should not cross the core as to interfere with placing the concrete and securing a solid core.

Figure A, type (1), shows one of the old forms of Hennibique type. In this type of column the principal reinforcement

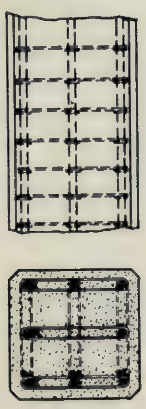


Fig. A. Type 1.
Old Style Hennibique Col.

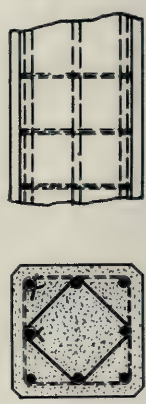


Fig. B. Type 2.
New Style Hennibique Col.

consists of heavy longitudinal bars tied across from one to the other by small ties. It will be noted that these ties cross and recross the core of the column, requiring considerable care in filling to make sure that there are no voids in the finished work.

Cases have occurred with this type where the concrete has been arrested in pouring part way down the column and on removal of the forms there was an open space of two feet or such a

matter between the concrete above and below and the load above carried only by the vertical bars. Evidently such an arrangement of metal is somewhat dangerous even though it may be properly executed and with unusual care prove satisfactory from the standpoint of strength.

Figure B, type (2), shows an improved form of vertical reinforcement with ties in which eight bars are used and the ties are placed in the form of squares, one inscribed within the other. The advantage of this type over that previously shown lies in the fact that the central core of the column or inscribed square is clear and unobstructed throughout.

Figure C shows a type of column reinforcement consisting of four vertical rods with wrapping or ties holding them together at intervals. This is suitable for very light loads where the

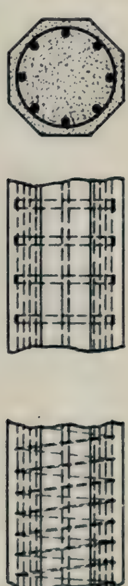


Fig. D
Considere type of hooped Cols. with independent bands and spirals.

Fig. C.
Suitable only for light Cols.

concrete is more than sufficient to take the entire compression without excessive stress.

Figure D, shows a column section of the Considere type in which the vertical rods are hooped with spiral reinforcement. Considerable work has been executed using the hooped columns

but omitting the vertical steel. This as the writer looks at it, is a very grave mistake.

Hooping may be of two types. First, a spiral coil in which the wire is wound around the core of the column in the form of a spiral and second, in which independent hoops are placed at intervals and attached to the vertical reinforcement.

There is a large amount of apparently conflicting experimental data regarding the strength of reinforced concrete columns. As noted in the writer's remarks on the subject of common theoretical errors he will in no wise consider the type of column with a single vertical rod in the center on the ground that such construction is radically dangerous and should not be used, hence need not be considered in a practical treatise on concrete-steel construction.

The strength of types A, B, C, and D all depend upon, first the strength of the concrete, second, the amount of vertical steel used and, third, the amount of ties or hooping holding the rods in position and bringing lateral restraint upon the concrete.

Theoretical formulae based on the ratio of the moduli of elasticity of concrete and steel cannot be depended upon for a satisfactory solution of the problem presented by the third element noted and we must depend largely upon experimental investigation to determine reasonable and safe practical values to use for our working stress.

In deciding upon these values we have to consider the column, first, from the standpoint of its ultimate strength in the finished building, second, from the standpoint of its strength and safety during construction and third, the relative values of the various types in securing strength at a minimum cost.

In order of merit these types may be rated as follows:

Type D, with a proper proportion of vertical steel combined with the hooping ranks first, from the standpoint of safety and economy.

Type B, second.

Type D, hooping but with no vertical steel third; and types A and C fourth.

It may be stated that type A is practically out of date and discussion concerning it may be eliminated.

For type C, the writer in his practice is willing to allow on a 1-2-4 concrete, 350 pounds per inch of the core area between

rods, 10,000 pounds per square inch on the vertical steel and the volume of metal in the ties to be treated as imaginary verticals with a working stress equal to that allowed for the vertical bars, the ties to be spaced not further apart than ten times the diameter of the vertical bars in case the bars are one inch section. Where smaller bars are used spacing not to exceed 9″ nor the section on the tie to be less than a one-quarter inch round.

Type B, concrete for the core area, 1-2-4 mix, 600 pounds per square inch, 10,000 pounds on the vertical steel, one and one-half times the volume of the ties treated as imaginary verticals. These ties should not be spaced further apart than 9″ and if to be considered of value not more than ten diameters of the vertical bars apart.

Type D, the Considere type, is by far the most economical type of column reinforcement that has been originated. This was brought prominently to the attention of the public by Armand Considere, an eminent French engineer.

The principle involved is this. By restraining the concrete laterally its strength in compression is greatly increased. Just as an ordinary piece of stove pipe filled with sand will carry a load a number of times greater than the pipe itself would be able to do, due to the fact that the metal is strained in tension while the filling, held in position by the restraint of the pipe, carries the weight of the load.

There have been quite a number of experiments on hooped concrete using only spiral hooping. In these experiments it has been noted that after the ultimate strength of the concrete has been developed, splitting and scaling of the outside shell occur, combined with a large vertical distortion and considerable lateral bending before the ultimate failure.

Evidently if we expect to develop the core of the concrete to a point beyond its normal strength we must prevent its distortion or bulging laterally and also the sliding or flow of the concrete between consecutive bands or turns of the spiral, hence a certain proportion of vertical steel must be used in connection with the hooping to secure the best results.

Since this type of column ranks first from the practical standpoint and in reality is the only one fit to use in general construction it is deserving of the most careful consideration.

We will commence by comparing the two types referring to

bands and spirals. The advantage of the independent band lies in the fact that it is not enlarged except by the outward pressure of the concrete under stress. The longitudinal pressure of the load with the spiral tends to shorten the spiral as a whole to close the turns together and incidentally to increase the diameter of the spiral just as a coiled spring when returned to its unstrained position, increases somewhat in diameter. This increase in diameter however is comparatively minute though it must be considered as one of the elements in weighing the advantages of the two. On the other hand the spiral can be easily made up, it is somewhat cheaper than welded hooping for light wrapping and is to be preferred where the hooping is not required to take extra severe stress. Where the hooping required, owing to the desire of the designer to use a small sized concrete column is heavy, then circular bands become the easiest to handle and the most desirable from the practical standpoint.

To be efficient, spiral wrappng should not exceed a four inch pitch when combined with not less than four vertical rods not less than three quarter inches in diameter for columns carrying moderate loads nor should the pitch exceed three inches if the full value recommended hereafter is to be used in considerng the hooping.

Where six or eight vertical bars are used not less than seven-eighths inches in diameter, circular hoops may be spaced six inches from edge to edge apart using the full value allowed. Where, however, the pitch of the hoops is increased the allowances for hooping should be decreased in proportion to the increase of the spacing, but should not exceed 9″.

Our standard allowance on a 1-2-4 concrete with this type of Considere column is as follows:

1,000 pounds per square inch on the core area, 12,000 pounds per square inch on the vertical steel bars and 16,000 pounds per square inch on the hooping treated as imaginary verticals having a volume of 2.4 times the volume of the hooping.

For these values the ratio of the length of the column to its diameter should not exceed twelve. For a longer column ample vertical steel should be provided to provide for flexure. Where the strain on the concrete developed by hooping exceeds 2,000 pounds per square inch the proportions of the mix should be increased from one cement, two sand and four stone to one cement, one and one-half sand and three stone and extra care

should be used in the selection of the stone aggregate to see that it is hard and satisfactory. Good screened gravel is a more satisfactory aggregate where high working pressures are used for the columns.

ARTICLE 2.

Safe Limit of Compression on Vertically Reinforced and Hooped Columns.

A wonderful degree of strength may be developed on a properly hooped and longitudinally reinforced concrete column and it becomes a question as to how great values it is permissible to use. The writer is inclined to place about 4,000 pounds per square inch developed pressure upon the core as the approximate limit with suitable design. Under these ultimate conditions he would investigate the column from the further standpoint that there should be enough vertical steel to carry the entire load at a little more than the yield point value say at 50,000 pounds per square inch. That there should be sufficient hooping to develop the value of the load figured at 40,000 pounds per square inch on imaginary verticals corresponding to 2.4 times its volume. That the gross area of the column inside of the fire proofing should be sufficient to carry the load at 4,000 pounds per inch.

Tests shown were made for the writer at Phoenixville, and represent results obtained by testing full sized columns. It will be noted that a reasonable amount of vertical steel was used, combined with the hooping, that the columns are practically all the same diameter with about 1-½″ of concrete outside of the hooping. With this proportion of vertical steel, cracking and chipping of the outside shell did not occur until the hoops were overstrained and stretched, and it should be noted the failure occurred by bursting the hoops, that the concrete was a good rich mix.

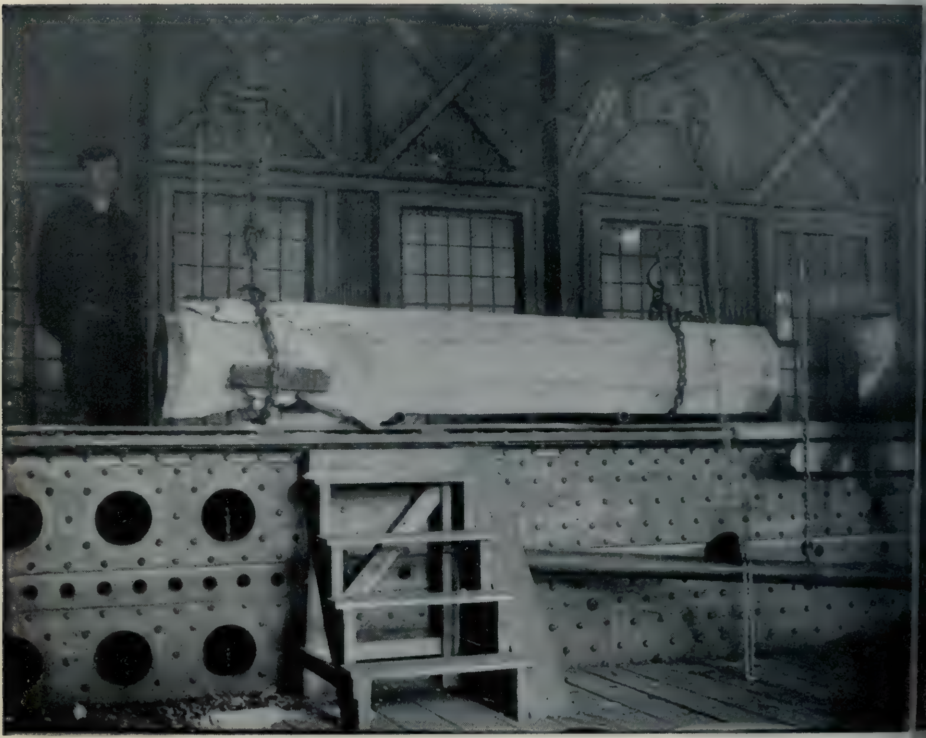
ARTICLE 3.

Partial report on tests on full sized columns, made at Phoenixville, Pa., for C. A. P. Turner, engineer, by Mason D. Pratt, M. Am. Soc. C. E., Harrisburg, Pa.

Test No. 1.

Marks on Column—None.

Reinforcement—Eight 1⅛ inch round bars vertically.



Test No. 1. Showing column as it came from the testing machine.

Band spacing—9 inch vertically.

Hooped with 7-32 inch wire spirals, about 2-inch raise.

Total load at failure, 1,360,000 lbs.

Remarks—Point of failure was about 22 inches from the top. Little indication of failure until ultimate load was reached.

Some slight breaking off of concrete near the top cap, due possibly to the cap not being well seated in the column itself.

Test No. 2.

Marks on column—Box 4.

Reinforcement—Eight 1⅛ inch round bars vertically.

Band spacing about 13 inch vertically, 14½ inch diameter.

Wire spiral about 3 inch pitch.

Point of failure—About 18 inches from top.

Top of cast iron cap cracked at four corners.

Ultimate load—1,260,000 lbs.

Test A. Column No. 4 after test.

Remarks—Both caps apparently well seated, as was the case with all the subsequent tests.

Test No. 1. Showing Column as it came from the testing machine.

Test No. 3.

Marks on column—4-b.

Reinforcement—Eight ⅞ inch bars vertically.

Hoops—1¾ inch by 3-16 inch by 14 inch outside diameter.

Band spacing—13 inches, vertically.

Ultimate load—900,000 lbs.

Point of failure—About 2 feet from top.

Remarks Concrete, at failure, considerably disintegrated, probably due to continuance of movement of machine after failure.

Test No. 4.

Marks on column—Box 4.

Reinforcement—Eight 1 inch round bars vertically.

Hoops—Spaced 8 inches vertically, 14 inches diameter.

Wire spirals as on other columns.

Total load at failure—1,260,000 lbs.

Remarks—First indications of failure were nearest the bottom end of the column, but the total failure was, in all columns, within 2 feet of the top. Large cracks in the shell of the columns extended from both ends to very near the middle. This was the most satisfactory showing of all the columns, as the failure was extended over nearly the full length of the column.

It may be noted that these columns are the lightest that have been used in the "Turner system;" that the hooping is lighter than that used with heavy loads, and that the strength developed indicates that the formula above recommended gives conservative results.

These columns were made by Butler Bros., of St. Paul, and the tests to demonstrate to the owner and the building department the conservatism of a design for a large reinforced concrete building for the Lindeke-Warner Co, of St. Paul.

The concrete mixture was one part Portland cement, one part sand, one and one-half parts buckwheat gravel, and three and one-half parts gravel ranging from one-quarter inch to three-quarters inch in size.

It should be noted that in these tests the cracking of the shell did not occur until the hoops were overstrained, and that the strength of the hooping closely defined the ultimate strength of the column with the proportions of vertical steel used.

ARTICLE 4.

Economic Column Design.

Figuring on the basis of the values outlined it is evident that the cost of the columns becomes less as they increase in size and the hooping and vertical steel is reduced to an amount which would render the use of 1,000 pounds per inch on the core permissible, that as we reduce the column diameter it is necessary to increase the amount of the steel and thus increase the cost of our reinforcement for the building.

This is a point not generally understood by contractors. One bidder inquires as to what size columns would be permitted by

the owner and figures on that basis. Another bidder makes the column larger by two or three inches, uses less steel and puts in a lower price on his reinforcing scheme, and the owner is frequently inclined to save the difference without giving the first bidder opportunity to revise his proposition on the second basis.

In general the variation in the percentage of steel may run from one per cent vertical reinforcement to ten or twelve, hooping from .3 to three per cent.

It should be noted that the efficiency of the hooping varies as D* while the core area varies as D^2, hence a small increase in the diameter of the core enables a large decrease in the amount of hooping and vertical steel, that the hooping costing 1.6 times as much as the vertical bars in a column in place has an efficiency 2.4 times that of the same cost of metal in vertical steel.

Now on the other hand, certain relative proportions between the amount of the vertical steel and hooping are necessary to secure the best results. For example, it is better to use no less than four vertical rods in any column three-quarters of an inch in diameter and a minimum of ⅜″ rounds for hooping spaced not over 4″ centers.

As the load on the column increases and the diameter is increased, the number of vertical bars should also be increased, the writer's common practice being to use about six in a twenty-four inch column and eight in a twenty-eight or thirty, more if it is necessary to keep the column small in diameter and carry a very high working load.

Figure 29 shows a column designed very liberally indeed and put up in winter for a working load of between one thousand and eleven hundred tons. This column has an extra large margin of safety and could readily be trusted with fifteen or sixteen hundred tons with a full factor of safety of three and one-half.

The volume of vertical steel for cases where over two thousand pounds per inch have been developed on the core of the column should not be less than one and one-half times the volume of the hooping and in developing a higher value, that further the value developed should not exceed the ultimate unit strength in compression of a 10″ cylinder of the concrete used six months old.

In general the size of the vertical bars should be limited to

[1]D is the diameter of core above notation.

one and one-half inches diameter and in the ordinary column should not exceed $1\frac{1}{4}''$ and generally not over $1\frac{1}{8}$.

In a tall, narrow building it is good practice to splice the columns by independent splice bars. When using the above values no columns less than $12''$ in diameter should be used. Vertical steel in the column should not be less than .7 of the volume of the hooping when $\frac{L}{d}$ equals five and the minimum should preferably be increased with the increase of this ratio, this increase being approximately one-third of the increase in $\frac{L}{d}$

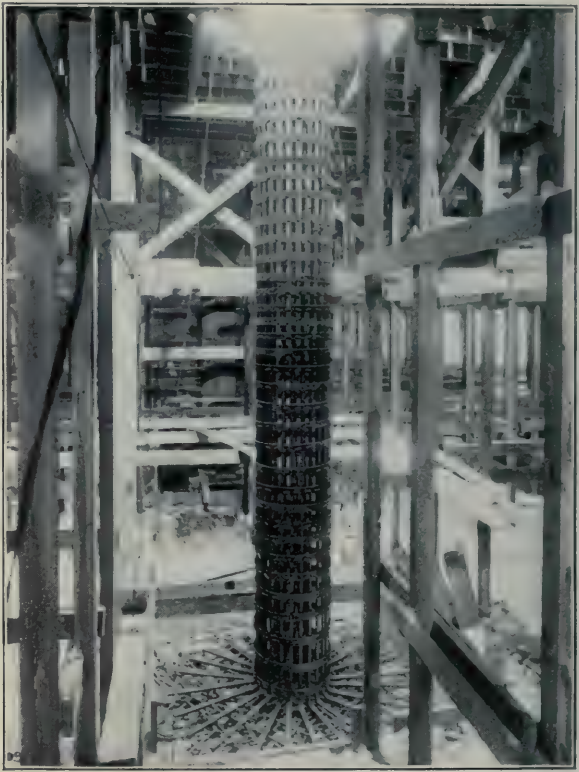


Fig. 29. Reinforcement of column in Hamm Brewing Co.'s Stock House. Carrying a load of 1,100 tons. Hoops 27 inches in diameter.

The Joint Committee on Concrete and Reinforced Concrete have allowed values for the best type of reinforcement about twenty per cent as great as the writer would consider permissible for the better types of hooped column reinforcement and one

hundred per cent higher than he would consider safe to entrust for results to the ordinary gang of workmen when using some inferior types of column reinforcement which are commonly used today.

From the experience gained from the construction under his direction and more or less closely under personal superintendence of several hundred thousand columns the writer's views are based on a fairly broad experience and he is of the opinion that the values recommended are eminently conservative.

To summarize the conclusions regarding economic column design we may say that the cheapest column to carry a given load is one in which the minimum amount of vertical steel and hooping consistent with safe construction and proper design are used. In other words, that concrete and a rich concrete at that is the cheapest material to carry a heavy load where the space occupied by the column is not too valuable as it is in the high office building. In the latter position heavy reinforcement, extra rich mixtures and high values are desirable and with proper design may be safely used.

Another point which it is well to call attention to, is the fact that it is better not to make the vertical reinforcement bars continuous. The writer prefers to splice them with each section of the column. The reason for this is indicated by the tests at Phoenixville which showed a failure of the columns two feet from the end wherever the cast plates bore directly upon the steel. Where the plates were raised and a slight thickness of concrete interposed between the ends of the bars and the cast iron plate failure occurred along the length of the column uniformly.

ARTICLE 5.

Uniformity of Concrete Columns Compared to Structural Steel.

The writer is firmly convinced from such tests as he has made using the largest machine in the country that there is a higher degree of uniformity in the values of reinforced concrete columns made with ordinary care and with well designed reinforcement than with the average structural steel column.

The reason for this opinion is that with the concrete column we have a solid core. The larger the column the greater the uniformity in strength—in strong contrast to some noted fabrications in structural steel.

With a structural steel column we have the uncertainty due

to the form, and make-up of the section. Thus in the majority of small struts and columns of structural steel and iron which have been tested we have a specific ratio of the area of the web and flanges in channels used in the make-up of the struts or a complete circular or box section as with the Phoenix column or with the box column of two channels and two plates.

The semi-empirical formula for struts has been worked out for the radius of gyration for these forms of sections and a large variation in the ratio of the area of the several parts or proportion of the section leaves the formula inapplicable to the section as was proved by the failure of the great bridge at Quebec. In this case the area of the flange as compared with the web was about one-tenth of the ordinary proportions between the area of the flange and web in the rolled channel and there was absolutely no experimental data in existence covering the lattice bar and secondary web stresses for such a combination.

In concrete work these secondary stresses the uncertainty due to changes of form are entirely eliminated by the uniform solid core. There can be no doubt as to securing a solid casting with ordinary care if the type of reinforcement selected is that recommended in this chapter.

Improper handling of the concrete might lead to greatly reduced strength of the section just as burning a piece of structural steel in forging will leave the piece weak and worthless. For example, the writer was called upon to inspect some coupler pockets made by a Pittsburg firm as the purchaser claimed that they were worthless. Taking a heavy sledge he struck the coupler pocket which was 4 by 1¼ in section, a sharp blow in the U breaking ten square inches of metal with a single blow of the sledge.

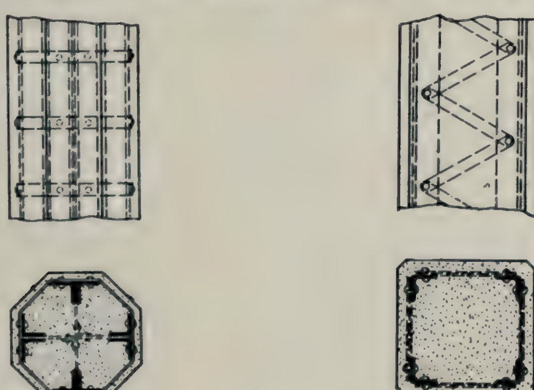
This is a fair indication of the uncertainties in steel construction where ordinary care has not been used and weak members can readily be fabricated in concrete in the absence of ordinary care and improper procedure in casting. Such for example as allowing the column box to fill with water, then dropping in the concrete, letting it sink through the water and allowing the sand and coarse aggregate to separate and go to the bottom or again, filling the beam box before the column is filled and allowing the concrete to flow diagonally and wash a little at a time into the column, separating out the inert material and finer particles of

sand in so doing and segregating them in the column thus securing a material in the core half the strength that it should properly develop. There is no excuse for this uncertainty, however, with proper conduct of the work and even at the worst the uncertainty involved is less than that in ordinary steel construction.

ARTICLE 6.

Structural Steel Reinforced With Concrete.

This combination is not well fitted to give good bond between the steel and concrete. Where a square column, latticed is used as in the McGraw building in New York, the writer would be willing to allow about 12,000 pounds on the steel in compression and 450 pounds on the concrete. Where the column is made up of eight angles, similar to the Gray column except that hoops are used in place of angle braces the writer would be willing to use the same value on the steel and about 750 pounds on the concrete.



Patented.

Structural steel columns reinforced with concrete.

Where beams are thoroughly encased in concrete there is no known method of figuring exactly how much this combination is good for but it would seem from tests in quite a number of cases that we may legitimately allow an increased strength of from fifteen to thirty per cent in the strength of twelve or fifteen inch beams thoroughly encased in concrete integral with a four or five inch slab at the top; the concrete being at least three inches above the top of the beam.

As the writer regards such an arrangement, in general uncalled for, and lacking in economy, he will waste little time in attempting to discuss it.

CHAPTER VII.

General Principles Governing Economic Design.

ARTICLE 1.

Method of Investigation.

Concrete steel construction even if we take simple elements at a time presents so complicated a problem that it can not be handled neatly by means of equations showing the cost of the variable items and determining their proper proportion by placing the first differential coefficient equal to zero in the usual mathematical manner. For that reason we must pursue our investigations more along practical lines in the following manner:

First, we must consider questions of general arrangement, including column spacing, then spacing of beams, choice of type of reinforcement, cost of centering for each kind and cost of aggregate, etc.

ARTICLE 2.

Column Spacing.

As regards column spacing, for light loads eighteen feet each way is economical with comparatively small variation in cost with an increase of column spacing up to 22 ft. For heavy loads, such as 350 pounds to 600 pounds, per square foot, column spacing seventeen to eighteen feet is a little more economical than longer spans.

ARTICLE 3.

Comparison of Types.

We have noted the coefficients of the bending moment and that the flat slab mushroom type has by far the smaller coefficient. This advantage however, is offset for light loads by the disadvantage of lack of depth d in the value of the moment of resistance. Further that the stiffness varies as d^2 but the deflection coefficient is very much smaller than for simple beams, hence we conclude that for stiffness under light loads a very low percentage of steel and a thicker slab in proportion than will be used for other types is desirable for the mushroom system.

Where the loads become very heavy then this system becomes more and more economical as the depth of the slab increases in view of the remarkably small coefficient of bending.

A little computation as to the cost of concrete and steel will show that quite a large variation from exact economic proportions of the two materials for strength will make comparatively little difference in the ultimate cost, while it does make a large increase in stiffness by the use of the smaller amount of steel and more concrete as has been pointed out.

The mushroom type has the advantage of simple centering and low cost of placing the steel and aside from that but little advantage over two way beam system for light loads but requiring much less material both in concrete and steel for heavy loads and moderate spans up to twenty-four or twenty-five feet and greater spans where the loads exceed 500 pounds per square foot.

The advantages of the two natural concrete types of building over type I of figure one, lies quite largely in the cost of centering. The centering for the mushroom system frequently costs as low as seven and eight cents per foot of floor including the cost of column forms as against fifteen and sometimes twenty cents for such as type I shown in figure (1).

Type II, with slabs reinforced in one way are at a disadvantage since the coefficient of bending is three times as high with one way reinforcement for the slab supported on two sides that it is for the slab supported on four sides.

Type II is frequently made with a combination of hollow tile and thin reinforced concrete beams between the hollow tile in order to get economic depth for long span slabs and to secure reasonable deflections under load. This keeps the cost of centering down approaching the cost of the mushroom system. It means, however, extra care in placing the steel and the labor of putting the tile in position and the cost of the tile.

Where the cost of the hollow tile is low, for light loads and long spans this type may become a competitor of a true concrete type, but the general opinion among the contractors with whom the writer is acquainted even under these favorable conditions seems to be the cost of labor combined with the cost of the tile amounts to a figure which exceeds that of the best all concrete type.

It has the disadvantage from the standpoint of safe construction in the lack of tying the work together in two directions and

the disadvantage of combining two materials having different coefficients of expansion and hence reduced ability to withstand high temperature stresses when exposed to the heat of a severe fire. This type will be discussed more fully under details of construction.

ARTICLE 4.

Centering.

Centering as we have noted is one of the important considerations, since the centering runs in point of cost from twenty-five to thirty-five per cent of the total cost of the construction. This item should be given the most careful consideration by the expert designer.

This subject is one that is best treated under the details of centering and will be left for discussion under this heading. In the selection of the type to be used by the designer however, this part of the problem of economic design must be kept clearly in mind.

ARTICLE 5.

Economic Make Up of Columns.

This has been treated in a general way under computation of columns but it may not be amiss to state that for the minimum cost the highest proportion of concrete consistent with sufficient vertical steel and hooping to render the construction safe beyond question in erection should govern in the design except that the space occupied by the columns is of more value than the increased cost of heavier reinforcement with the reduced size or diameter of the columns.

ARTICLE 6.

Economic Design of Beams.

We have noted that the coefficient for bending of a continuous beam is two-thirds as great at the support where the beam is well restrained, and one-third as great at the center as in the case of a simple beam. Now for safety, we need to have ample lap of the bars, hence by carrying a part of the flange reinforcement up over the support we may double up the sectional area of the bars at the support and by carrying them to about the point of contraflexure or sufficiently far so that the negative moment in the case of a one panel load will be taken care of by slab reinforcement parallel to the beam we have need (considering moment

only) theoretically of two-thirds the section of steel for about one-third of the length and one-third of the section of metal for two-thirds the length of the beam of that required for the simple beam. In other words we have the following comparison from the standpoint of theoretical economy. That the metal required for a continuous beam is one-half that required for a simple beam and further that the construction with a continuous beam is safer to erect since the work is more securely tied together and it can be depended upon not to fail suddenly but only by the actual stretching out of the metal to the point of ultimate fracture with a good concrete in case of loading equal to three or four times that which it was calculated to sustain.

This theoretical economy however cannot be fully realized. Two-thirds in place of one-half would be nearly the limit attainable.

Evidently the greater the depth the less steel that will be required. This condition however, in the ordinary building is determined generally from the standpoint of appearance and the extra cost of walls for a given clear story height rather than the theoretical economic proportions of steel and concrete.

A mistake which is frequently made is to build the beams very narrow and deep especially where they are spaced closely. Such construction is lacking in resistance to high temperatures since too great an area is exposed and should preferably be avoided on that account.

A minimum width of ten to twelve inches for a reinforced concrete beam should be observed in a building that is intended to be fireproof to a high degree and such a width for moderate spans of sixteen or eighteen feet will generally give ample concrete to properly surround reinforcement in the beams.

In general there should be sufficient width to allow one inch of concrete between the bars or a width not less than one and one-fourth times the diameter of the bar if the bars are parallel for any considerable length. Where they are bunched as at the top of the beam and there is ample spread beyond this point in the beam this requirement becomes of no especial importance.

Economic proportion of concrete and steel in the beam would be approaching the point where the cost of the steel and the cost of the concrete would balance, to the extent that the amount of the steel increases with the depth of the beam and also the amount of concrete.

However, in the T beam which is the general case with which we have to deal in the ordinary building this percentage would be based upon the area of the beam below the slab plus the area of the slab each side of the rib which it is permissible to consider a part of the compression flange of the beam and economic proportions would be with a smaller amount of steel, since the amount of the slab figured in with the beam is not added material as far as the beam is concerned and hence the comparison should be based more properly upon the area of the rib below the slab of the concrete added to form the beam hence the economic proportion of steel would in general be reduced under those of the limiting proportions fixed to secure conservative working stresses upon the concrete. This is modified to the extent that the cost of centering increases with the increase of depth of the beam. These practical considerations seem to have been entirely overlooked by Capt. John S. Sewell, in his paper presented to the American Society of Civil Engineers in 1906, on the subject of economic construction of reinforced concrete floors.

ARTICLE 7.

Economic Paneling.

We have discussed economic column spacing for the flat slab and column construction and we have yet to discuss the economic beam spacing where beam and slab construction is used in the general types, as one and three.

Type III, as a beam and slab type is most economical for long panels. Where the columns are spaced so that the panel is rectangular or nearly so, the economy of this type is greatest. In general it is not desirable to depend on two-way beam system where the shorter side is much less than six-tenths the longer side. For such cases the writer prefers if the beam and slab type is to be followed to divide the panel by a beam framing into the longer beam at the center, thus leaving approximately square slabs in the panel.

For beam joists and slabs the spacing of ribs is governed somewhat by the character of the centering used. If the centering is arranged in panels eight and ten-foot spacing of beams may be worked out quite economically where the beams rest on walls.

Where, however, they frame into beam girders this increased cost in framing should lead logically to the adoption of longer span slabs and fewer ribs.

In general, however, no exact rule can be given covering all cases as the conditions of the problem, size of the building and the manner in which it divides up for the purpose intended fixes so many of the conditions that a fair degree of experience combined with practical judgment is sufficient to decide in the majority of cases without computation.

ARTICLE 8.

Bearing Walls or Full Concrete Skeleton.

A very important question in economic design is the question as to whether bearing walls are to be preferred to a full concrete skeleton.

For a low building such as four to six stories bearing walls are generally cheaper than a full concrete skeleton. For buildings higher than six or seven stories a full concrete skeleton with curtain walls costs less than with heavy bearing walls.

It will be noted in the illustrations in this book that this point has been followed by practical designers. In putting up a concrete skeleton the additional cost involved in making provision for two or three additional stories is so slight that we generally advise the owner to make this provision if there is a reasonable probability that he may use the additional floor space to advantage in the future.

ARTICLE 9.

Concrete Exterior or Brick Exterior Walls.

In some cases where the concrete aggregate is cheap and union brick layers' wages are high it is better to use a concrete exterior wall. Generally, however, exterior walls may be constructed much more cheaply of brick or material that can be laid up without the necessity of using forms as the forms for exterior wall construction run into money quite rapidly.

ARTICLE 10.

Rich Mixture.

Economic construction in reinforced concrete requires a rich

mixture. This is a necessity, first, from the standpoint of certainty of computation, second from the standpoint of quick hardening which enables the early removal of the forms and from the standpoint of economy due to the fact that we can use less material of good quality which we can absolutely depend upon than we can of a material of an inferior quality and uncertain character which is liable to be discredited due to its slow hardening through the lack of necessary amount of cement. Further where the material used has been of good quality the construction can be increased in strength to any desired degree by the addition of more good concrete though the strength so secured will not be at as low a cost as if the original design was for heavier construction.

ARTICLE 11.

Honesty True Economy.

A corollary to this proposition is this that true economic construction requires an honest mixture of the materials with no skimping of the cement and hence the successful contractor in this line of business is the one who is absolutely honest and reliable since skimping almost invariably leads to the detection and discredit of the perpetrator in the early stages of the work after the concrete is cast.

ARTICLE 12.

Economy in Selecting the Aggregate.

Good bank gravel when obtainable makes an excellent concrete. In using it if it is uniform in the proportions of coarse and fine material these should be determined by screening out the sand pebbles which are under ⅛″ diameter. Treating this as sand the proportion of the cement to the sand should be one cement to two sand for reinforced work or richer perhaps for columns. Then the mixture will stand four parts of stone but very likely in the ordinary run of gravel there will not be exceeding three parts or even two and one-half of coarse stone then should the proportion of cement be kept in proportion to the sand and while this may take an extra sack or sometimes two sacks per yard of concrete, the extra cement should be used provided that the additional cost is less than the expense of handling and screening the gravel and mixing in exact proportions.

Sometimes crushed stone or gravel is not available and a good hard smelter or furnace slag may be secured. This should be examined for chemical impurities which might injure the cement. The majority of furnace slags, however, make a good aggregate.

ARTICLE 13.

Cinders.

Cinders are sometimes used as an aggregate for concrete. Cinder from the soft Iowa coal is generally very injurious to the cement. In fact it may be stated as general principle that the only cinder fit to make a permanent concrete is that which is a hard or more or less vitrified clinker such as generally results from burning soft coal with a mechanical stoker. Too great care cannot be exercised in this respect as upon the character of the aggregate and its freedom from sulphur or other injurious chemical elements which would injure the cement, depends the permanence and integrity of the work.

In general a clinker concrete should not be used where a high degree of strength is required. It is desirable to use it for such work as roof work where the spans are short and it is desired to nail a tile or slate cover to the concrete roof slab. For such purposes the concrete should not be mixed too rich, otherwise it will be difficult to nail into it.

ARTICLE 14.

For strip fill stone or gravel concrete is preferable to cinders if the filling is to be counted upon as a part of the finished slab. Where a good concrete strip fill is used covered with ⅞ maple finished floor it is permissible to figure from five to six-tenths of the thickness of the strip fill as effective concrete in figuring the deflection and strength. When this is taken into consideration the concrete should preferably be the same mixture as the slab. With one-way reinforcement strips should be parallel to bars.

CHAPTER VIII.

Systems of Reinforced Concrete Construction.

ARTICLE 1.

Early Pioneers in the Field.

In taking up the various systems of concrete steel construction the name of Francois Monier, a French gardener, stands pre-eminent as a pioneer in this field, while to Hennibique in France and Ransome in the United States due credit should be given as leaders among the early pioneers in this line of work. To Armand Considère belongs the credit of the invention of the safest and most conservative type of column design, that is, the hooped and vertically reinforced column.

ARTICLE 2.

Monier System.

In 1867, Francois Monier obtained his first letters patent on reinforced concrete construction and subsequently built many water tanks, water mains, sewers and even houses in "Armored Concrete," as it is termed quite largely among the French.

While Monier was not an engineer but a French gardener his patents are somewhat broad in scope and his invention was taken up and pushed by G. A. Wayss & Company, of Berlin, and others.

The reinforcement of slabs consists of rods crossing one another at right angles and tied at their intersections. Originally flat floors were supported on steel joists in various ways. Another type which was used was that of the arch in which similar reinforcement was placed in order to provide for temperature and shearing stresses. The system is used quite largely today in the construction of slabs and barrel arches where the reinforcement is placed in two planes, one above the intrados and another near the extrados.

ARTICLE 3.

Hennibique System.

A great impulse to concrete construction was given in '92 when Francois Hennibique in conjunction with licensed contractors all over the world constructed a large number of structures in armored concrete valued at many million dollars.

Hennibique structures were a success from the beginning, standing all tests prescribed by municipal authorities and specifications of engineers and soon by their strength and low cost found favor and were adopted in a large amount of important building work. Figures 32 and 33 are typical of the Hennibique beam.

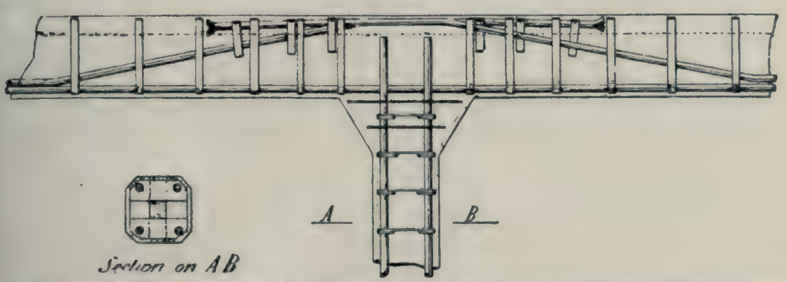


Fig. 32. Connection of a Continuous Girder to a Column. Hennibique cols. and beams.

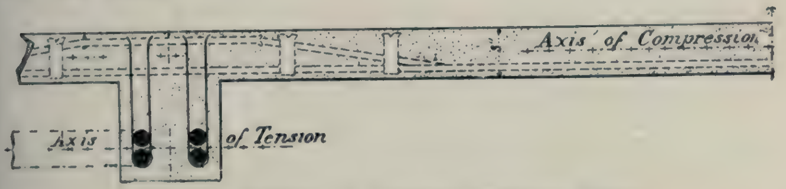


Fig. 33. Section of Armored Concrete Girder.

In considering the success of Hennibique it is natural to look for the reason in the characteristics typical of his designs. These may be summarized as follows:

1. Conservative end bearing for beams.

2. Ample lap of rods over supports combined with end anchorage.

Thus it will be noted that in the Hennibique designs the work is well tied together.

3. The bending up of a part of the main flange reinforcement in a manner that enables it to act in resisting vertical shear at the support.

In the stirrups used by Hennibique, they were spaced in a manner which gave them good anchorage at the top of the beam and kept them thoroughly surrounded with the concrete in strong contrast to some of those that have been employed by later constructors.

Hennebique Columns.

The earlier type of Hennibique column was sometimes found defective on removal of the forms. In the figure it will be noted that the vertical bars are tied together by punched plates which sometimes failed to be thoroughly surrounded by the concrete in casting and caused trouble. The later Hennebique type which is designated as type 2 in our discussion of columns, there has been comparatively little trouble with. On the other hand it is lacking in the economy of the Considère type and deficient in that degree of certainty with which the Considère column reinforcement can be counted upon to be fully filled with concrete giving a sound casting.

The typical manner in which the old style Hennebique column fails under test is suddenly and without warning by bulging the rods and crushing the concrete generally on 45 degree lines. In the later Hennebique type if ample metal is used in the ties it behaves more like the Considère type, although not as economical.

ARTICLE 4.

Ransome System.

This system, originated by Mr. E. G. Ransome, was one of the first to be introduced in the United States. The reinforcements used were bars square in section, cold twisted. It was one of the early attempts to secure mechanical bond. Figure one, type one, is a typical Ransome design.

The general characteristics of the system have been generally the use of the joist of concrete spaced closely together supporting a thin floor slab.

Another type largely used by him consisted in dividing the floor into square panels from seven to eleven feet square with beams running both ways. In other respects, the system bears some similarity to the Hennebique.

The centering in the Ransome system is evidently much

more expensive than that of the Hennebique type while the twisted rods add an expense of about two dollars per ton to the cost of reinforcement.

The claim for these is that the cold working raises the yield point of the metal enabling the use of higher working stresses. On the other hand, it is well known to the structural engineer that all cold work done upon material raises the yield point at the expense of the resilience, making it brittle and less reliable and for that reason the writer believes that cold worked material should not be employed in reinforced concrete construction, although hot twisted bars cannot be considered open to the same objection.

Again, as to the bond; in the construction of something like six hundred acres of reinforced concrete floor in all types of buildings the writer has failed to discover the necessity for greater bond than that secured by the use of a rich concrete and his preference is to make sure that the concrete end of the combination is first class when the question of bond will settle itself..

ARTICLE 5.

The Unit Girder Frame.

The unit girder frame shown in figure 34 consists merely of a slight modification of the Hennebique beam in that the

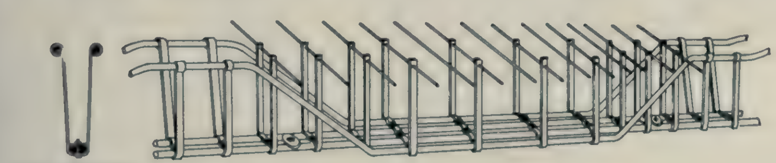


Fig. 34. Unit Girder Frame.

stirrup is an inverted loop attached to the rods at the bottom instead of lapping over them. It is essentially the same principle and is lacking in the excellent continuity feature of the original from which it is copied.

ARTICLE 6.

Cummings System.

The Cummings system of reinforcement originated by R. A. Cummings, of Pittsburg, is illustrated in figure 35.

The following features are especially worthy of notice:

1. Make-up of the beam.
2. Clips for supporting the slab reinforcement.
3. Make-up of the column.

The beam is made up quite largely on the order of a simple beam in which the rods are carried up diagonally from the lower flange and looped, the section of the steel reducing from the center towards the end of the beam.

The general theory of the design of the beam is along the

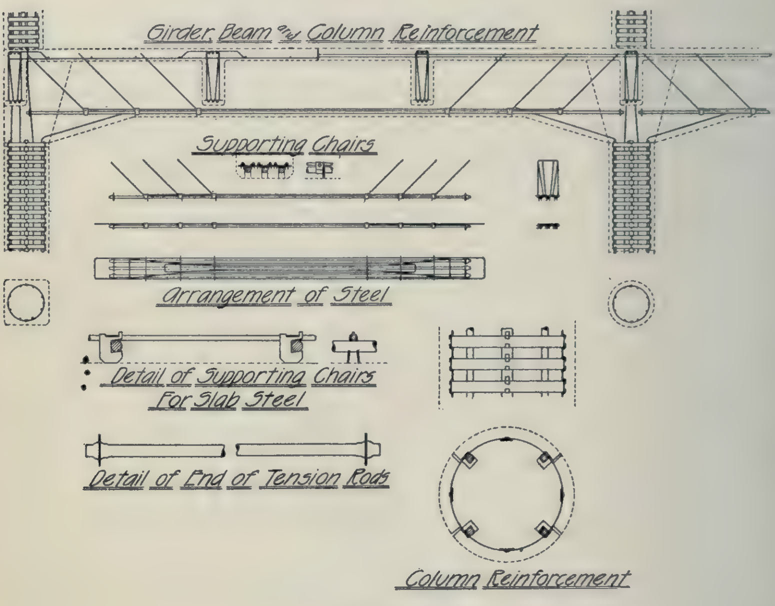


Fig. 35.

line of analogy with the Warren truss. Unlike the Kahn system of reinforcement to be discussed later, the diagonal tension member is carried near the top of the beam and looped which gives it a positive anchorage in the top portion, thus carrying out logically the assumed analogy.

Tests made at Champaign, Ill., show up this beam in a very favorable light as a simple beam. The reader is referred to the bulletin of the University of Illinois No. 29, on tests of concrete beams, series of 1907-8.

When used as a continuous beam in a building a rod is placed near the top of the beam, causing it to act in the nature of a partially continuous beam. The distribution of the reinforcement then is not altogether logical for the development of the continuous beam but as some of the building ordinances are framed this type meets the requirements in a highly satisfactory manner.

The Cummings clip for supporting the slab reinforcement is a simple, inexpensive and ingenious device for clamping the rods of a two-way system together in a very satisfactory manner. It is quite clearly shown in the accompanying cut.

Column Reinforcement. In the early design of the Cummings column electrically welded hoops were arranged closely spaced on spacing bars, the hoops attached to the spacing bars by staples. In his earlier work he seems to have placed almost entire dependence on the hooping without using what the writer would consider a desirable proportion of vertical steel, but in some of the later designs this criticism would not hold good.

Where the columns join the floors as shown in the accompanying sketch the writer would not consider the concrete well restrained for high pressure due to the lack of continuity of the banding.

Mr. Cummings considers that this lack of continuity is offset by the enlarged section of the concrete which he provides, as shown in the accompanying detail.

With a two-way main beam system of type III this criticism as to the joint of the column would have little weight, but where girders and lighter beam joists are used the writer would not consider the detail as outlined in the accompanying sketch satisfactory.

ARTICLE 7.

Kahn System.

The Kahn system has but one distinctive feature differing in any wise from previous construction in this line, that feature is the Kahn bar. The bar is rolled in the form of a square with two flat projections, one on each side. These projections are cut across transversely, sheared longitudinally and bent up forming the attached web member bar or fin bar as it is known.

This bar is illustrated in figure 36, taken from one of the Kahn System advertisements. Examining it closely it will be noticed that the length of the fins are generally exaggerated in these cuts as will be shown by trying to describe a circle from the point at which the fin is supposed to be sheared to the end of the fin as shown in the cut, when it will be noticed that the fin is shown twenty or thirty per cent longer than could be sheared from the bar, thus giving an exaggerated idea as to its length.

In theory the Kahn bar is supposed to act with the concrete after the manner of a Warren Truss, and proof of this theory as advanced by the advocates of this type of reinforcement, reminds one of the story of the friendly discussion between two lawyers, in which the question came up as to who was recognized as the most prominent attorney in the place.

"I am of course," said the first. "How can you prove it?" asked his friend. "Why, I do not need to prove it, I am willing to admit it," replied the first.

Thus the advocate of this type of reinforcement apparently

Fig. 36. Kahn Trussed Bar, Alternating Type.

advances similarly convincing proof that the bar acts in the manner claimed.

Comparing the fin with the Cummings loop it will be observed, first, that there is no positive anchorage in the upper portion or compression area of the beam; that the entire dependence is placed on the adhesion between the fin and the concrete for a very short length of fin. Second, that as usually made the fins do not reach the upper or compression portion of the concrete.

Third, that at the end where the shear is a maximum the fins are necessarily shorter than towards the center where the vertical shear is much smaller in amount.

No series of tests have been made which indicate in any wise that this bar is the equal to its weight in plain bar reinforcement properly and scientifically placed. In making this

statement the writer does not wish to be misunderstood as asserting that there have not been good and serviceable buildings put up with this type of reinforcement. On the other hand, as this type has been too frequently designed there has been used insufficient lap over supports, which, combined with the lack of provision for shear at the end where the shear is a maximum, has resulted in a number of serious failures.

We illustrate herewith the detail of the beam used in the Bixby Hotel at Long Beach, Cal., and the accompanying figures

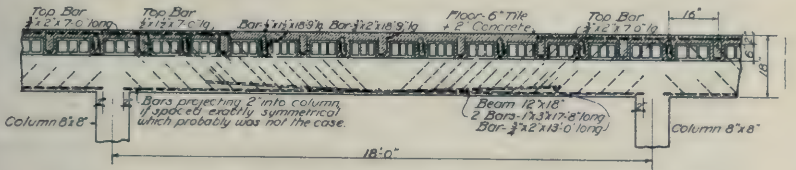


Fig. 37. Detail of beam in Bixby Hotel, Long Beach, Cal.

show the failure of the main beams by shear and the total collapse of a large section of this building.

Referring to figure 38, failure of the beams by shear at the point of connections to the columns is well illustrated, while in figure 39, the columns at the right and left of the center are intact and the beams seem to have broken away in a similar manner. Figure 40 also clearly indicates this failure of beam by shear at the column connection and the general inefficiency of a rod at or near the top of the beam at the support in taking care of this shear unless the rod be bent downward and continued from column to column.

In the writer's judgment had the centering been left in place in this building for a period of six months it would have stood and there might have been no serious trouble with the building. On the other hand, as an engineer, he cannot regard that kind of design as legitimate which must necessarily be treated with this extreme degree of care, since there is no need of designing in a manner which would leave an opportunity for such a sudden and complete failure.

In referring to the beams in this building it will be noted that the length of bars for upper stories were insufficient to have lapped over the column more than two inches, provided they had been spaced exactly symmetrical about the center of the beam. This was probably not done by the workmen and

Failure of Bixby Hotel at Long Beach, Cal.

Fig. 38.

Fig. 39.

the entire resistance to shear then became a question of the shearing value of the concrete section and the insignificant resistance offered by the adhesion of the green concrete to the downward projecting fins of the short bar placed in the top of the beam over the column in an effort to secure partial continuity.

A similar collapse with loss of life occurred with a design presenting the same defect at Rochester, N. Y., in a building for the Eastman Kodak Company. In this building in addition to the bad feature noted in the design of the beams the columns

Failure of Bixby Hotel at Long Beach, Cal.

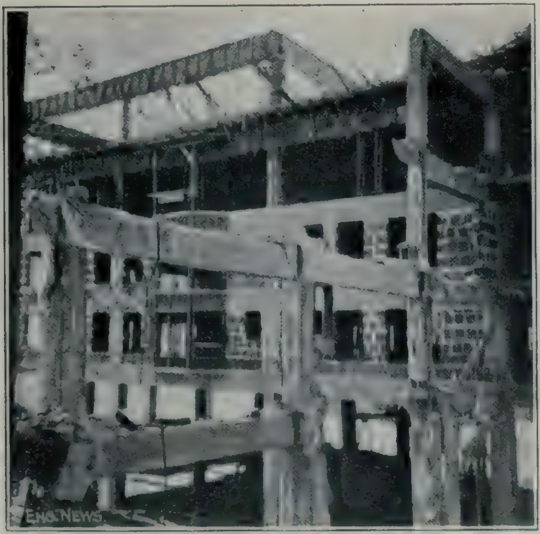


Fig. 40.

were constructed with four of these fin bars, the inwardly projecting fins tending to prevent a solid casting, resulted in weak columns as well as beams lacking in strength in the respects noted.

View of the column and beam which failed in this building is shown in figure 41. More complete description of this failure will be found in the Volumes of the Engineering News of 1906.

As in the case at Long Beach, it is unquestionably a fact

that had the centering been left in place eight or ten weeks longer no failure would have occurred.

Figure 42 shows a test which was afterwards put upon the roof of this building near the point of failure on that portion from which the centering had not been removed and which was given more time in which to harden. This feature has been covered in a report by Mr. Edwin Thacher, which will also be found given quite completely in the same volume of the Engineering News.

A distinct disadvantage of these bars lies in the fact that

Fig. 41. Failure of Eastman Kodak Co. building at Rochester, N. Y.

they are flat and hard to get thoroughly surrounded and covered completely with concrete. They are somewhat more difficult to handle in erection than plain rounds.

From the standpoint of economy a fair estimate of the shop cost of cutting, over and above plain rounds, would be in the neighborhood of one-half or three-quarters of a cent per pound, hence it we rate their efficiency at ninety per cent that of plain

bars properly arranged add thereto the cost of shop work and a fair estimate of the selling cost and cost of advertising we should be forced to rate them from the economic standpoint as fifty to sixty per cent as efficient as their equivalent value in plain material.

That the representatives of this system have profited by the Rochester failure seems evident by the fact that many of them claim that they are no longer pushing this type of column but they are using a conservatively designed column of the Considere type with spiral hooping and vertical reinforcement.

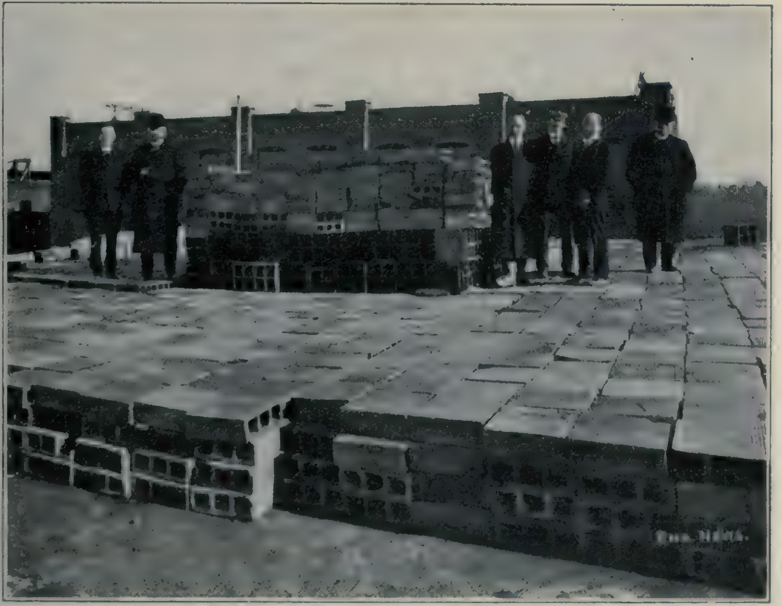


Fig. 42. Test on roof of Eastman Kodak Co. building made after the concrete had been given more time to harden.

ARTICLE 8.

Imitations of the Kahn System.

As the Kahn or Trussed Concrete Steel Company, by exceptional push have succeeded in introducing their type of reinforcement in many buildings over a wide area, there have been quite a number of attempts to imitate the features of their design, namely the attached web member.

Quite a few schemes have been gotten out which are even worse than the original, but they have met thus far with but little favor.

The Gabriel system is another type which seems in a measure an off-shoot of the Kahn system. An endeavor to provide a main line of reinforcement with some kind of a web member.

In general detail this follows the old idea of DeVallière, as illustrated in the work on reinforced concrete by Charles F. Marsh. The writer can see little advantage in this general idea.

Monolith reinforcement is another and perhaps better attempt

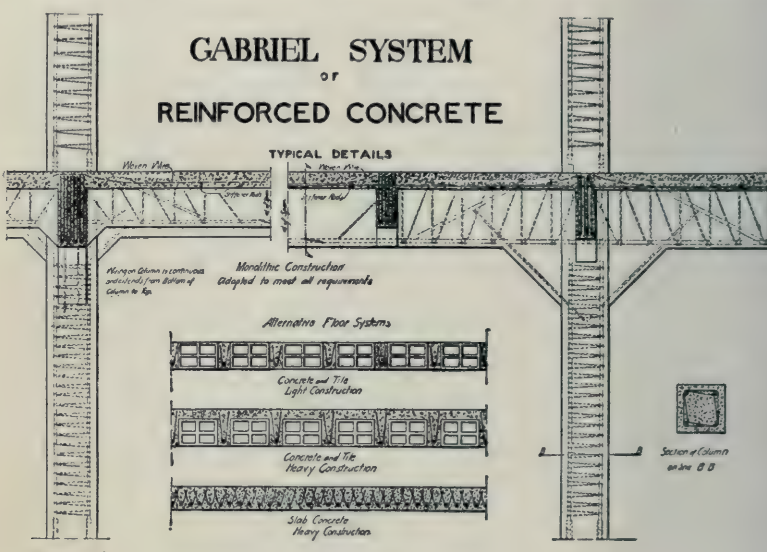


Fig. 43.

in this line. In this the web members are attached in the form of loops to a bar somewhat in the shape of the English bullhead rail and clamped thereto by pinching the flanges of the heavy member together making a rigid attachment to the smaller shear loops.

This partakes more of the nature of the Cummings beam while lacking its economic feature if regarded as simple beam reinforcement. As noted, however, in the earlier part of the work, the writer considers true continuous beam reinforcement as the better type in building construction not only on the ground of safety but also on the ground of economy.

ARTICLE 9.

Pin Connected Girder Frame.

This girder frame is shown in figure 44. It may be regarded as a very good beam reinforcement. It has, however, all the disadvantages of structural steel construction in that the length must be determined exactly and that there is a large

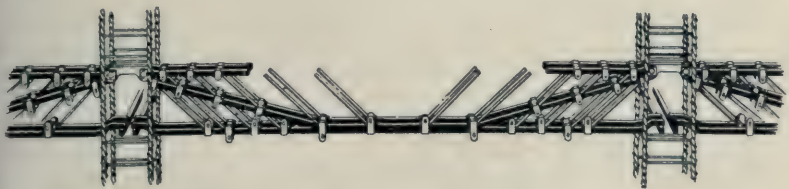


Fig. 44. Pin-Connected Girder Frame.

amount of needless shop work. Again, as sometimes made up, the packing and clamping of one bar upon the other when these bars are flat leaves a chance for the continued deterioration of the metal due to the lack of opportuntiy for the concrete to thoroughly surround the steel where one flat bar is clamped on the top of the other.

The column shown in this cut and too frequently adopted are the older style Hennibique instead of the Considère type.

ARTICLE 10.

Wire Cable as Reinforcement.

A number of attempts have been made to use cold twisted wire cables as reinforcement and quite a large amount of work has been executed with this material in connection with wire fabric. The general idea of these systems has been to put an initial strain on the cables, in other words, draw them taut before casting the concrete. This has been done by the use of turn buckles and wall anchors.

Quite a number of cases have come under the writer's observation where the walls were pulled out of line and still more where there has been a gradually developed sag in the concrete due apparently to the stretching out of this twisted wire member. The writer's general observations warrants him in advising the constructor in this line to leave this type of reinforcement severely alone if he assumes responsibility for the work, particularly where there are long span slabs.

Another device concerning which he can speak in no more favorable light is an attempt to use chain reinforcement, not only on the ground of economy, but failure of the reinforcement to pull in line in a satisfactory manner.

ARTICLE 11.

Expanded Metal Reinforcement.

In a previous chapter we have discussed to some extent the value of two-way reinforcement and expanded metal, although a rough sheared and much abused metal fabric in a measure fills this requirement when the long way of the mesh is in line directly between supports.

This fabric will develop with a 1-2-4 mix nearly double* what we would figure it capable of when considering merely its cross sectional area. Turned in the reverse direction however, and it is very weak indeed, due to the fact that the consecutive strands do not pull in line and do not present stiffness due to their position when strained in that manner.

They are now making sheets of expanded metal up to thirty feet in length. The weak feature of adopting this material lies, first, in the laps required where the spans are large.

Second, due to the fact that a rough sheared fabric is not as the writer regards it a suitable tension member when we can readily get bars rolled to uniform section without flaw and at about one-half or one-third the expense per pound and secure the same advantage without the objectionable laps by proper arrangement of the bars in similar diagonal directions.

The sheets are, however, very convenient to handle for short spans and in some kinds of work where the price justified their use the writer has employed this material and secured satisfactory results.

ARTICLE 12.

Wire Fabric.

In taking up the question of wire fabrics we have to consider, first, those which are woven and partake of the somewhat objectionable features of the twisted cable in the strand. Second, those which have the straight strand with lighter diagonal web mesh. Third, those which are electrically welded.

* See Chap. IV, Art. 5 and Chap. III, Art. 15 for arch action.

While it may be possible to weld the fabric electrically without hardening or causing the metal to become brittle near or in the approximate vicinity of the weld, the writer's observations have not satisfied him that this is the case with the majority of the fabrics on the market and his preference is for those which are woven for this reason.

For light slabs, roof work and the like, for wrapping beams and many details of reinforced concrete requiring light reinforcement, the various fabrics on the market form the best possible type of metal to use since while their cost is somewhat higher per pound than rods, convenience in placing and handling may offset this additional cost and the thorough dissemination of the metal through the concrete in small units is conducive to better results than the use of the same sectional area in larger units less uniformly disseminated or distributed.

In nearly all of the writer's work where he has been called upon to put in roof slabs on a structural frame and light work of various kinds he has found it advantageous to employ this material and believes that the best results can be secured with it under these conditions.

Many of these fabrics, especially those having lighter gauged wire, are galvanized. This is an unnecessary precaution if the fabric is well housed and due care taken to see that it is thoroughly encased in the concrete. On the other hand, it is a fact that in view of the small section of the wires a little rusting would so reduce these sections that the precaution taken to protect the metal before it is finally imbedded in the concrete may be a wise one, particularly where there may be a question as to the degree of care with which the contractor may handle or house the material.

ARTICLE 13.

Independent Beam Construction.

The large percentage of the cost involved in the centering for reinforced concrete has been productive of many attempts to reduce the cost by manufacturing independent units in the shop in such a manner that they may be shipped to the work and set in place just as timbers in ordinary mill building construction.

We illustrate herein in figures 45 to 49, inclusive, series

Fig. 45. Showing independent beam construction.

Fig. 46.

of views of collapse which occurred with this type of construction. The building is a design which was passed upon by the building department of the city of Chicago and unquestionably was figured in accordance with the building laws of that city while failing completely to meet ordinary common sense requirements for safe construction.

Figure 45 shows the under side and general features of this construction. It will be noted that the frame proper is a structural frame encased in concrete. Second, that the beams used are independent T beams, that lugs are cast to make a full bearing at the ends. The reinforcement of these T beams consists of rods at the bottom and bent rods somewhat similar to the Hennibique type.

Figure 46 shows the saw-tooth roof of this building in which the cover consists of these beams placed upon a structural frame encased in concrete. The beam partially covered is shown at the left. Close examination of the independent beams on the slope at the right will indicate the small loops of steel in the form of a staple projecting above the beam by which they were handled and placed in position.

There was no reinforcement in the top of these T beams hence it was necessary to handle them right side up with care since the slightest inversion of the beam would crack it by its own weight. The space between the saw teeth of the roof was to have a cinder fill and the usual coating to shed the water. However, before this roof coat was completed the rain thoroughly soaked the cinder fill, resulting in a collapse of a large section of the building.

Figure 47 is a view showing a section of the side wall of the building. The lack of bond between the concrete beams and the wall is quite evident from this view. The beams being merely set on a corbel built out from the side of the wall.

Figure 48 is another view of the collapsed portion of the building in which the greater portion of this miserable corbel is well illustrated, also the fact that the concrete had scaled completely from the side of the double channel structural main beam which is hanging down in an inclined position.

Figure 49 shows the sag of these beams as placed on the saw teeth and a portion of the collapsed building.

It is extremely unfortunate that the watchman in this build-

ing was caught and smashed in the fall of the material. This is but a fair indication of the total lack of judgment on the part of not a few of that class of architects who are willing to assume that they are experienced in this line of construction

Fig. 47. Failure of a Chicago building constructed of independent beams.

without training or practical experience in building and are willing to take any kind of a chance.

They consider that this type of construction requires no brains and no experience because it is simply a question of the combination of mud and steel.

The fact cannot be too strongly emphasized that the science of correct and economic construction in reinforced concrete is a specialty in itself. It is a business that cannot be learned in a day or a week, nor can it be learned from text books alone.

Fig. 48. Inside view of walls after collapse. Independent beam building.

The engineer who is a safe man, and should command the confidence of the investor, is he who is thoroughly familiar not only with mere theories, but with all of the details connected with the execution of the work, and who has studied carefully

the strength of numerous constructions put up under different conditions at different temperatures, and knows with certainty the capacity they can be depended upon to develop in the actual test of the finished work.

There are many who have an idea that if they get the steel into the concrete somehow that is all that is required, while as a matter of fact upon the position and arrangement of the reinforcement in the concrete, the strength will readily vary from one hundred to one thousand per cent, and there is perhaps no line of engineering which requires greater skill and good judgment to get the best results for the money expended.

Fig. 49. Showing lateral sag of independent roof beams.

In selecting this example of idiotic construction the writer may not have done justice to the possibilities of independent beams. In Switzerland some little success has been made in casting hollow beams, reinforced at the four corners, which could be handled and placed without danger of cracking and breaking the construction before it was secured in the building.

The radically dangerous feature of placing the entire weight to be carried on the corbel outside of the face of the wall is a method frequently attempted by insulating engineers in order to get the insulating material back of the concrete. The writer's

only recourse in dealing with this kind of architect has been to write a letter calling attention pointedly to his view as to the stability of the construction as they had outlined it and put himself on record that he would not assume any responsibility whatever for the stability of the construction if carried out along the line which he was requested to follow. This has invariably brought about the desired result, permission to so place the concrete that it had a proper bearing and could tie the building together in a satisfactory manner.

In general, any type of independent beam construction is totally unsuited for a building of any considerable height. It lacks the monolithic features of true reinforced concrete. It may be expected to crack the plaster at each and every joint and from the standpoint of cost there is no reason for its existence or adoption in practical construction. It may be observed that this building which failed complied strictly with the building ordinances in vogue in the city of Chicago and is a fair indication as to the weakness of the ordinances of more than one large city and the dangers to which such ill-advised, ignorantly drawn rules subject the owner, the workman and the general public.

They are very particular in the city of Chicago about whether the deflection of a properly designed slab is 1-700 or 1-600 of the span when the said slab can carry six times its working load and they will swallow a dangerous piece of construction of the character outlined for the reason that it complies literally with the wording of their badly drawn ordinances. Unfortunately it is expected in not a few cities to remedy the dangers incident to ill-considered laws by low working stresses for all designs, good, bad or indifferent, instead of getting out rational building regulations which would exclude dangerous types of construction and allow legitimate and reasonable values in proper construction in this excellent type of building which properly executed and intelligently designed is the best and most permanent in existence.

ARTICLE 14.

The Mushroom System.

The Mushroom System is so-called from the peculiar formation of the rods around the column head and from the remarkable rapidity with which it may be erected. The idea of this system is primarily to simplify the centering and thus reduce the cost of the temporary part of the construction without skimping the materials in the finished work.

The arrangement of the reinforcement is designed with a

Exterior of Lindeke-Warner building, 235x165 feet. Turner "Mushroom" System; costing $10,000 less than timber mill construction for this building. St. Paul Minn.

view of securing the maximum efficiency of the materials through straining the concrete in a number of directions, the compression in one direction tending to balance and off-set that in another; incidentally to concentrate the maximum amount of rein-

forcement around and over the support where the shear is the greatest, to enable removal of the forms at the earliest possible period, and finally to eliminate beams and ribs which interfere with light and catch dust, cost money to plaster and finish, and reduce the clear story height. The flat ceiling so obtained gives free and unobstructed illumination from the windows; it permits one to place partitions anywhere without regard to the floor, giving an unusual stiffness and solidity due to the fact that a part of the material which in the beam type is placed in the ribs, is consolidated in the slab, making the slab of unusual thickness with an actual decrease in the total amount of material where the loads are at all heavy.

From the nature of its reinforcement, this system, as reported by consulting engineer Mr. John Geist, of Milwaukee, is particularly well fitted to be used in the construction of warehouses, and similar buildings where, due to the presence of aisles or passageways for trucks between the columns, just where the mushrooms with their heavy reinforcement are placed; and it is rather an encouraging fact that the heavier the loads to be carried, the more economical this form of construction becomes, as compared with the usual beam-and-slab method.

Illustrations of this type of construction will be given on a number of the following sheets; among them the enormous wholesale building for the Lindeke-Warner Co., St. Paul, on which Butler Bros., of St. Paul, bid $10,000 less for the reinforced concrete on this system than they did on the architect's design for timber mill construction; Louis A. Lockwood, of St. Paul, was the architect.

In the wholesale hardware house for the Bostwick-Braun Co., Toledo, Ohio, designed for working loads of 500 pounds per foot on the lower floors, and 350 pounds on the upper floors this system effected an economy of $30,000 over more common types. This construction was adopted by Architect Geo. S. Mills, after a thorough investigation of its merits by Engineer Geo. V. Rhines.

In fighting a fire where the contents of the building are particularly inflammable, those who have given the matter attention will understand how a rib in the ceiling stops the stream of water which is elevated sufficiently to strike the ceiling, whereas, given a flat ceiling it is easy to reach any part of the floor

Interior Lindeke-Warner building, St. Paul, Minn. Mushroom System.

desired, as if the stream is elevated and strikes the ceiling it merely glances along and is spread over the floor where required, instead of being stopped short where it strikes a projecting beam or rib.

Adaptability. The Mushroom System has been used advantageously for nearly all classes of buildings—court houses, state capitols, warehouses, factories, hotels, bridges and high office buildings.

In the office building partitions can be readily shifted to suit the tenant without interference of rib framing, while cost of plaster finish is but 65 per cent that where beams are used, and for a given clear story height, there is a saving of 10 per cent of exterior walls.

In a factory building the Mushroom System possesses the following important advantages:

1. A better distribution of light.

2. The flat ceiling enables more convenient placing of shafting than where ribs are used.

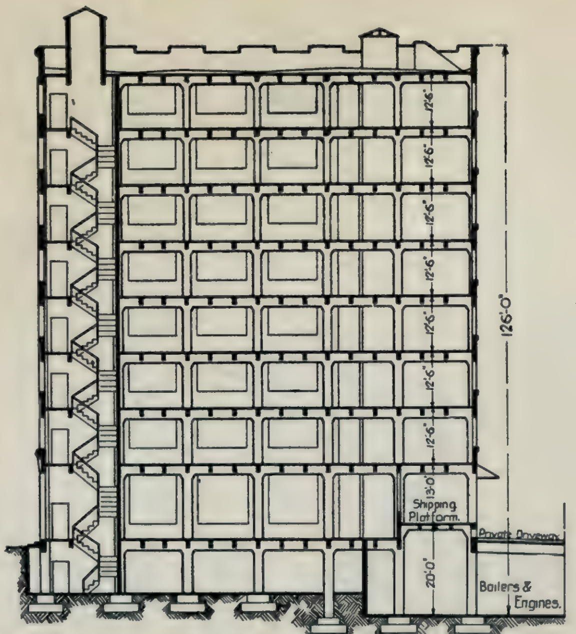
3. Freedom from vibration. Owing to the fact that any concentrated load brings into action the reinforcement of the entire panel, it has no local effect and consequently the construction is more nearly free from vibration than any type of construction being put up today.

We illustrate a plan of the Dill & Collins building in Philadelphia as originally designed in beam system, and changed to Mushroom System on account of these advantages.

Limit of Spans. The Mushroom System has been built and tested for nearly all spans from fourteen to thirty feet, and larger spans can readily be made if desired, at increased cost.

Notes as to Cost of Mushroom System. For warehouse purposes, where the capacity of the floors is from 200 to 300 pounds per square foot and column centers 18 feet to 20 feet, the Mushroom System is as cheap as timber construction in the central portion of the United States. Where the loads are heavier than those noted it costs less.

Economic Column Spacing. For loads of 100 to 150 pounds per square foot, column spacing from 18 feet to 22 feet, center to center, is most economical. For loads of 300 to 500 pounds, spacing 17 feet to 19 feet centers is preferable, while for 500-



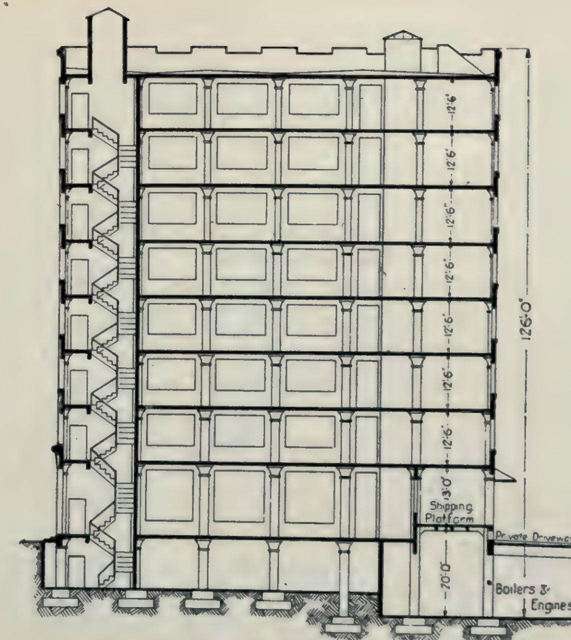
Section of Building.

This System abandoned for Mushroom.

Fig. 1. Beam and girder system. Note projecting beams and girders.

Clear height from top of finished floor to bottom of girder, 10 feet 1 inch. See how the beams and girders interfere with the distribution of light.

The story heights are the same as in Fig. 2. Compare this cut with Flat Slab System, Fig. 2.



Section of Building.

This System Used.

Fig. 2. Mushroom System.

Clear height from top of finished floor to ceiling line, 11 feet 6 inches. No beams or girders to interfere with light, with shafting or piling of goods

The story heights are the same as in Fig. 1. Notice superiority.

pounds loading there is small increase in cost for 22-foot spacing.

This system was originated by the author with the idea primarily of simplifying the centering and getting up a construction which could be used nicely for light loads. As he began to investigate the properties of the flat slab he became more and more convinced that instead of a system adapted to light loads it was essentially more economical for heavy loads.

Its advantage from the standpoint of economy increases, the heavier the weights to be carried, for the reason that the increase in thickness of the slab in view of the low coefficient of bend-

Typical Mushroom Floor. Bostwick-Braun building, Toledo, Ohio.

ing gives it very material advantage over any type of beam construction for moderate spans. Where loads are very light the quantities of material can be kept down for a beam system nearly equal to that for the mushroom system and it has then only the advantage of simplicity of centering combined with a somewhat thicker slab than is usually used making it more nearly sound proof from floor to floor since the material which in other systems would be placed in the beams is consolidated in the slab.

In patenting the system the author did not confine himself to specific details but he selected rather claims covering as broad-

ly as possible the continuous flat slab and column construction as he was the first in the field as far as he is aware, to work out a construction commercially feasible of this character.

ARTICLE 15.

Turner System of Beam and Slab Construction.

Beams for this type of construction are shown in figure 53, are well adapted for use in the construction of type III.

Provision for the bending moment over the support is usually taken at $\frac{W\,L}{12}$ and the sectional area is secured by lapping the larger portion of the bars over the support at the top. Thus in ordinary spans five rods are used, two at the bottom of the beam and three bent rods. Two of the bent rods dip downward toward the point of contraflexure, while the third is bent downward to the center of the beam and following along the same lines after lapping over the column into the adjacent span.

With slab reinforcing in two directions the idea is that the slab reinforcement supplements the beam reinforcement over the support and enables the main lines of reinforcement to follow such directions as will enable them to take care of the major portion of the shearing stress in the beam. To this end the rods are carried up in direct line over the center of the support and dip down beyond the support. With this arrangement the vertical component of the strain in these bars due to the bend upward can act directly in resisting vertical shear.

By lapping the beam rods over the support there is a very large section of metal around the column capable of carrying shear. Thus there are in section on four sides of column forty main beam rods acting in shear, an amount of material sufficient to make a very rigid connection indeed. In fact, in one building in which this type of connection was used the contractor put in a footing which was not in accordance with the plans and considerable settlement resulted. The author was able to jack the construction up, raising it and relieving the beams from strain gradually then putting in the footing in a proper manner. The beams were afterwards tested to double the working load and stood the test without cracks or other indications of weakness.

Figure 54 shows the operation of jacking up the column.

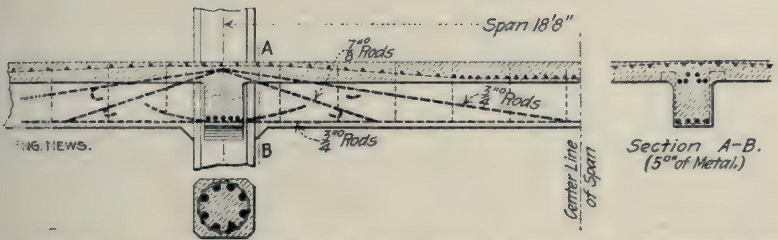


Fig. 53. Typical beam in new Aikens building, Winnipeg, Man. J. Woodman, Architect; C. A. P. Turner, Engineer and Contractor. Patented July 6, 1909.

Fig. 54. Jacking up column, after settlement of footing without material injury to the construction.

This experience was sufficient to convince the writer that where proper attention is given to the details of the work that reinforced concrete can stand an amount of distortion equal to or greater than that which it is possible for structural steel construction to withstand without serious injury.

The beam rods of this system are always bent on the work, the bending costing about $1.50 per ton. The writer's practice has generally been to use netting, wrapping the beams rather than the use of stirrups except as the stirrups serve as a convenient means of keeping the slab rods at the desired elevation over the beams.

The bending moment in a continuous beam, regarded as fully restrained at supports, is $\frac{W\ L}{12}$ at the support and $\frac{W\ L}{24}$ in the center. Supposing that we lap the rods over the support one-sixth of the span in length; then theoretically we would need two-thirds the section for one-third the length and one-third of the section for two-thirds the length that would be required for simple beam construction. We have in addition the fact that the maximum strain in tension occurs at the support where three-fifths of the main bars at their point of maximum stress gives a large vertical component capable of resisting vertical shear directly. Unfortunately we have to provide in addition to the condition of uniform loading for the condition of maximum live load covering a single beam or panel. This condition requires a greater section of metal at the center of the beam. It also requires provision where the load is heavy for negative bending moment in the top of the beam at the central portion. The latter is taken care of by the slab reinforcement where two-way reinforcement is used and our net economy in material over that which would be required for simple beam reinforcement is probably in the neighborhood of twenty-five or thirty per cent with the further advantage of securing a type of construction which is well tied together, in which the shear is well taken care of at the support where the maximum sectional area of reinforcement occurs.

Carrying the bottom rods of the beam across the support provides also steel reinforcement helping to reduce the compressive stress on the concrete in the bottom of the beam at the support and the lever arm of the beam is with this system taken

directly as the distance center to center of the steel rods at the support.

No system of construction which the writer has examined affords so large a section area of metal at and around the supporting column as is employed in the mushroom system and in the beam system above outlined, and its effect in practice is to secure practical protection from serious accident and failure during erection. Something like six hundred acres of these two types of construction having been put up without failure or accident chargeable to the risk of erection of concrete construction.

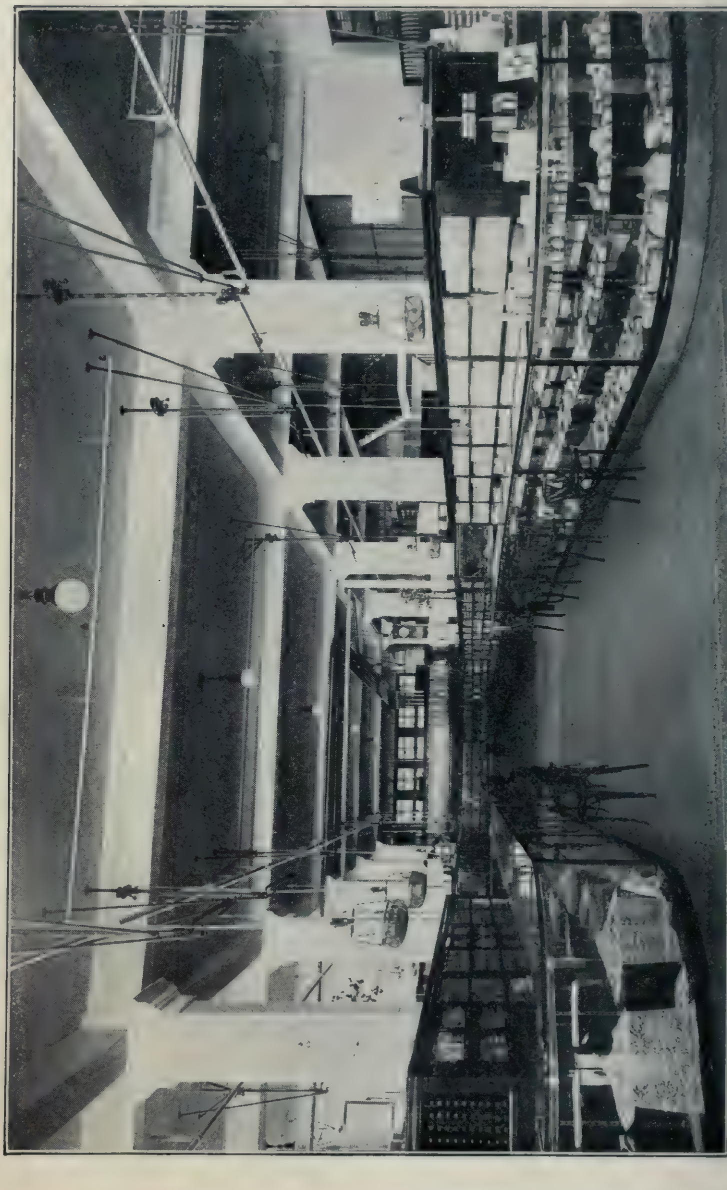
The writer has endeavored to work with only first class and reliable contractors. On the other hand, his conservative instructions regarding the removal of forms have frequently been disregarded.

At the time that the failure occurred in the Eastman Kodak building, the writer had a contract for the concrete work of the Aikens building in Winnipeg, and the foreman, contrary to instructions removed the centering seven days after the concrete was cast and the weather had been quite frosty at the time. The slabs were about nineteen feet square and five and one-half inches thick. The result was a sag of from ⅝ to ¾ of an inch in some three or four panels. There was considerable grumbling by the owner and his architect inspector until after the panels were plastered when this defect was no longer visible.

While such procedure in executing work is inexcusable the difference in the deportment of the material due to the manner of reinforcement contrasts strongly with that at Rochester, New York, where the forms were left in for three weeks and a bad collapse occurred upon their removal.

The writer has of late years used very little beam and slab construction for the reason that the flat slab and column construction has proved more economical and its form more satisfactory to the majority of his patrons.

For simple beams the author in his practice prefers to follow an arrangement of tensile reinforcement somewhat similar to the ties of the Bollman truss, and while such reinforcement is not quite as economical in point of weight as that of the Cummings system beam it renders, as the author would look at it, the construction somewhat safer for the early removal of the



Interior view of Smythe block. Showing dry goods department of Innes & Company, Wichita, Kansas. Reinforced concrete. Louis Curtiss, Architect; Martin Carrell, Contractor; C. A. P. Turner, Engineer. Turner System beam construction.

forms, and the fact that the rods are readily bent on the ground, that there is no shop work, is in its favor from the standpoint of economy in cost which tends to offset its disadvantage in point of weight.

As previously noted, however, the simple beam is employed only rarely in the ordinary run of building work. At least ninety-five per cent of the construction being of a character which enables the employment of the economic continuous beam or flat slab construction heretofore outlined.

ARTICLE 16.

Brayton System.

L. F. Brayton, of St. Paul, endeavored along in 1905 to develop a system having for its base the use of light structural beams which were to be reinforced with concrete and made to act in unison with the concrete by looped straps of light bands. The general idea was to enable the early removal of forms using short spans between joists. It did not, however, prove a commercial success and he shortly afterwards turned his attention to a plain rod system in which was employed columns of the Hennebique type and beams which were somewhat between the Hennebique and the Turner system beam. Stirrups used were similar to those employed by Degon but without the top reinforcement used by the latter.

Mr. Brayton has put up a considerable amount of very satisfactory work but the methods of figuring as given in a pamphlet called Brayton standards, unfortunately follows the lines of the ordinary treatises and are not based on tests of construction but rather on formulae copied from the early run of works on this subject.

ARTICLE 17.

Combinations of Reinforced Concrete and Tile.

The Faber system is well described in Kidder's pocket book. In this system reinforcement in two directions is generally employed. The hollow tile for this system is very substantially made, the opening in the center being circular in form and the end joints between the tile are usually left about one inch or so apart and pasteboard thimbles inserted in the openings in

the tile joining two consecutive blocks so that when the grout is poured between the tile a good end bearing is secured between consecutive tile sections in both directions.

Spans up to forty feet by eighty feet have been executed successfully in Germany with this type of floor.

As it seems to the writer the combination of tile and concrete is somewhat open to criticism from the fireproof standpoint on the ground of the difference in the coefficient of expansion of the two materials entering the composite structure. On the other hand, viewed from the standpoint of a non-combustible building and the ability of the construction to withstand perhaps a high degree of heat for a short time, the most which might be expected to occur with many classes of buildings, this type of construction may be considered satisfactory provided it can fill the bill from the standpoint of commercial economy.

Johnson System. In this type of construction fabric imbedded in a thin layer of concrete at the bottom of the slab and the ordinary hollow tile are laid thereon with narrow joints between and an additional thickness of concrete is placed on top.

For light loads this has been used up to twenty or twenty-four feet. In some localities where the cost of hollow tile blocks is low and concrete aggregate high, it may become a competitor of true reinforced concrete types. The *Kahn System* is also used to considerable extent with hollow tile. The beams between the tile being about one and one-half inches thick and a single rod in the beam.

A very large amount of this construction has been put in where the conditions were favorable from the standpoint of cost of the hollow tile block.

Among the combinations of hollow tile and concrete the Faber system appears to rank first from the standpoint of safety of erection and conservative design while there appears to be little difference in the other types except as regards the conditions in individual cases as to the amount of the end bearing secured, and whether the support is the narrow flange of a beam or channel of insignificant width or reasonably broad wall areas.

ARTICLE 18.

Deformed Bars as Reinforcement.

As noted in an earlier chapter, the writer prefers deformed bar reinforcement where the concrete is under water. Thus we may use plain rounds bent to give secure anchorage or twisted bars or bars rolled with some deformation such as corrugations, bulb and the like.

The common forms are the Thacher, the Corrugated and the Twisted bars and there are many others all working on the same general idea of securing some projection which will act as a mechanical bond. There is practically little difference in their efficiency for those classes of work in which this type of reinforcement may be advantageously used.

The little variations in form and contour possess a greater efficiency as talking points for the man selling than any point of practical value in the actual work. In reinforced concrete work each and all of these types possess a distinct disadvantage to which attention has not been heretofore called in the literature on reinforced concrete.

Whenever the centering is removed from the work at too early a period a slight deflection or sag results in case the concrete is not thoroughly hardened throughout. This is accompanied by a slight slipping of the bar. If the bar is a plain round bar the necessary readjustment between the concrete and the steel takes place with but slight disturbance of the concrete around the bar and when it ultimately hardens the shrinkage grip of the concrete around the steel is practically equal to work which has hardened under more favorable conditions as determined by numerous tests.

In the case of the deformed bar, however, the projecting lugs will tear through and disturb the concrete around the bar to a considerable extent and the concrete cannot then be expected to ultimately set up around the reinforcement in an equally satisfactory manner. Since the bond between the two materials in concrete hardening in air and with a rich concrete is largely of the nature of a shrinkage grip sufficient to develop the value of the bars in a comparatively short length, the objection noted is in the writer's judgment sufficiently serious to bar their use except the centering is allowed to remain in place longer than is conservative with plain bar reinforcement, since some chance

of slight deflection is commonly taken in keeping down cost by rushing the work.

A further objection to deformed bar reinforcement is that the cost of placing this class of bar is considerably higher than that in handling plain rounds in multiple way systems as the records of several of the writer's associates clearly indicate. The price asked for deformed bars generally ranges from two to three dollars a ton over and above the market price for their equivalent in plain rounds. Further, the quality of metal worked into a large part of the deformed bar reinforcement is old rail stock and other classes of brittle low grade metal.

ARTICLE 19.

Some Objectionable Beam Details.

The accompanying figure 55 contains a somewhat objectionable feature in beam design in the following respects: First, the

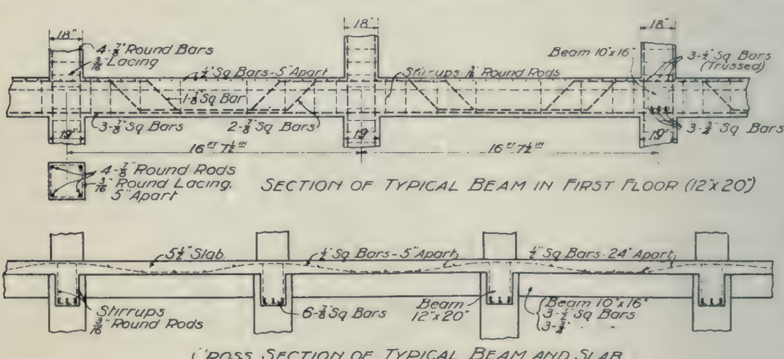


Fig. 55.

beams are designed as continuous beams while there is no reinforcement in the central part of the panel in the top flange to

Fig. 56.

Fig. 56. Shows a form of beam presenting a similarly objectionable detail as regards the bend of the rod.

take care of the negative bending if one panel is loaded and two adjacent panels unloaded. Nor is there any slab reinforcement parallel to the top flange of the beam to make this good.

As we look into the form of the main reinforcing bars we note that they are carried near the top flange outward from the support and then bend downward sharply. This kink in a main rod tends to add to the shearing stress in the concrete of the beam at the point of this sharp bend and in the earlier stages of the construction immediately after the removal of the forms it leaves the concrete subjected to excessive shearing stresses due to the sharp bend instead of assisting the concrete as it would do were these lines of reinforcement carried up at an easy angle toward the support.

ARTICLE 20.

Trusses in Reinforced Concrete. The writer has seen many crude details worked out for application of reinforced concrete to trusses. He is inclined to take the view that compression members of moderate span trusses can be economically fabricated in reinforced concrete but that the details of the connections to be satisfactory from the engineer's standpoint must be of a more substantial character than those that have been worked up in the few examples of practical work heretofore executed.

The Visintini System represents an attempt to fabricate independent trusses as girders or long span slabs. The joint details as usually worked out on this system hardly commend themselves to the structural engineer, while such details as were employed by Considère in his remarkable little truss bridge of seventy feet span would not be safe to trust to the execution of the ordinary gang of workmen. These defects in point of satisfactory connection between compression and tension members are in the writer's judgment capable of satisfactory solution and we may expect some development along this line in the near future in combinations of concrete and steel.

CHAPTER IX.

Foundations.

ARTICLE 1.

Bearing Values on Soil.

In a building the floor loads are carried to the columns or to the walls in case bearing walls are used, and the weight is concentrated on small areas of ground at the footings of the columns and the walls. Evidently if we are to avoid settlement the weight must be distributed over a sufficient area. The following are suggestions for the loading for material which can be clearly defined.

Ordinary ledge rock, such as good shale, lime stone and the like, twenty to thirty tons per square foot. Granite, trap where the ledge is not shattered, fifty to seventy tons per square foot. Hard pan, seven tons per square foot. Gravel, five tons. Clean, coarse sand, four tons. Fine sand, with a little clay, three to three and one-half tons. Hard clay, three tons. Clay, such as is to be found in Regina, which is rather soft, not over one and one-half tons. Blue clay of Winnipeg, two to two and one-half tons.

In each case, however, it is well for the engineer to look into the conditions carefully unless thoroughly conversant with the locality. His judgment as to the bearing power of the soil should be checked, if doing business in a strange city, by a careful examination of the buildings resting on similar foundation and general inquiry from the fellow members of his profession.

This caution may prove of value to the engineer doing business over an extended area, particularly if he is acting in a consulting capacity for a contracting firm assuming responsibility for the design.

In cases where there is filled ground, marsh quick sand and the like it is frequently necessary to use a pile foundation or distribute the weight over the entire area. In general, in reinforced concrete construction it is economical to use a thin footing and thoroughly reinforce it and to make the cncrete a rich mix.

ARTICLE 2.

Column Footings and Method of Figuring.

Figure A shows the form of footing which, as the writer looks at it, is somewhat objectionable for the reason that as usually placed the concrete is worked rather dry in order to make the slope without top forms, and the average contractor does not get the material in place so that it can be depended upon with certainty.

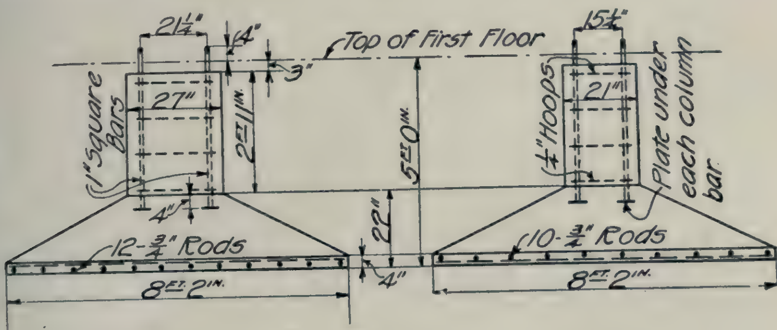


Fig. A.

The form which the writer prefers to use is shown in figure B, in which the footing is made in two layers The bottom

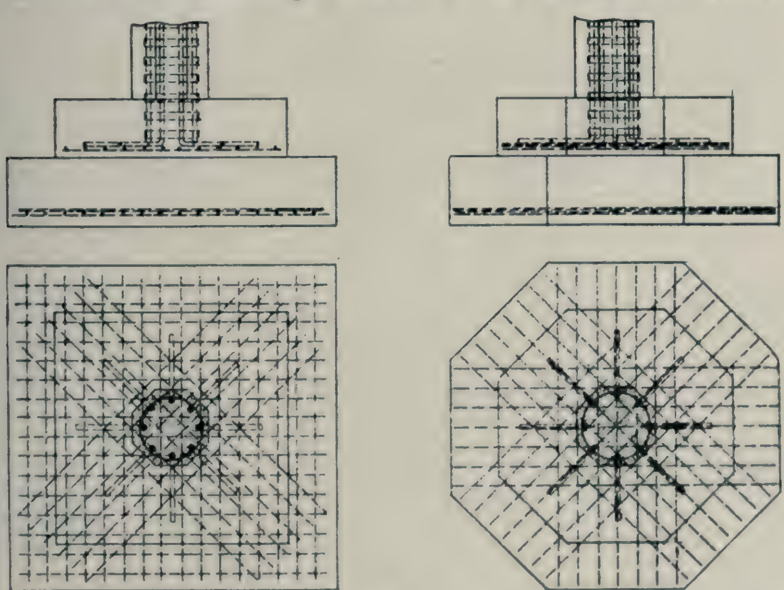


Fig. B.

layer cast with the rods at the bottom, then the column assembled and the top layer case with the column. Its advantage from the practical standpoint is first, that the upper layer of rods assist the footing in resisting the shearing strain of the column tending to punch through the footing and it assists also in distributing the load out over the lower plate. The bottom plate can be cast with a wet mix and we can depend with certainty on the bond between the steel and the concrete by the shrinkage of the wet mixture.

In computing the lower plate, the cross sectional area of the steel should be such as to provide for a bending moment of the upward pressure on the under side of the plate distributed uniformly over it with the column at the point of support.

The center of gravity of the area or pressure may be taken roughly as five-eighths of the half diameter of the footing, hence the actual bending moment is 5-16 Wd.* The metal acting as a flat plate would be as we have shown for the square panel twice as efficient as single way reinforcement and as the sectional area of each bar comes into play on each side of the center of the footing we can use four times the area of all rods crossing the footing times .85 of the total thickness of the two layers times the working stress on the steel as the resisting moment.

The amount of steel placed in the top layer is more a matter of practice than that of exact computation. We generally make the rods the same size as those used in the lower layer and waste no time in computing the stresses thereon. We prefer to take long rods and bend them hair-pin style for the footings rather than to use short rods.

There is a further advantage in ordering the rods this way, that in case the steel for the footings is delayed we can use the rods elsewhere when supplying the immediate need from stock.

ARTICLE 3.

Pile Foundations.

Where the piles if used will be continually wet and there is no possibility of changing conditions from that to alternate drying out to wetting there is no type of reinforced concrete pile that can compete with the timber pile in any locality where the

*d in this notation is the diameter or width of footing.

writer has been called upon to do business. However, where the piles are liable to be above the permanent water line or it becomes necessary to excavate thereto then and there concrete piles become an economic method of building up the foundation.

Practical concrete piles may be divided into two classes, one, those which are made up first and driven afterwards, and second, those in which a core or form is used and the hole filled in with concrete reinforced or otherwise.

Coming under the second class are the Raymond and Simplex piles. In the Raymond pile the core is driven, a thin cas-

Fig. 57 Driving corrugated concrete piles.

ing of steel left permanently in position and the concrete is deposited therein. The Raymond piles are made tapering to secure greater resistance against settling.

The first form of concrete pile made up and then driven, Hennebique first used for sheet piling as well as for bearing piles. In driving it is customary to use a jet loosening up the sand or earth at the bottom and rapping or jarring the pile into place.

Figure 57 shows a driver handling one of the corrugated piles. Figure 58 shows the cushion cap used in driving the pile.

Corrugated piles have been patented by Frank B. Gilbreth. The strong claim advanced in their favor is on the ground that the corrugations assist in jetting the piles into place.

The Simplex pile casing is driven, and as the casing is pulled up, the concrete is deposited and rammed in place, forcing it out to a somewhat greater diameter than the shell that has been driven.

The merits of the three types would apparently vary, dependent on the conditions. A pile on the order of the Raymond pile should be most suitable for a clay soil where the consistency or cohesiveness of the clay is such that the pile core can be driven and the shell omitted.

Figure 62 shows the Raymond pile expanded ready to be driven. Figure 63 shows the core in the leaders with the shell on the right. The core is expanded when driven and collapsed to be withdrawn from the shell.

Figures 1A and 2A and B show the get-up of the Simplex pile. With the steel casing is driven a point either of steel or concrete and afterwards the shell is gradually withdrawn and the hole filled with concrete as the shell is filled up.

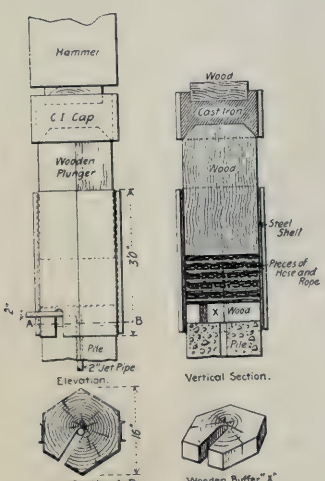
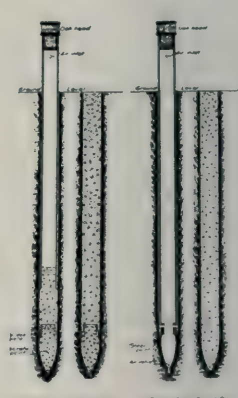


Fig. 58. Cushion cap used in driving the corrugated concrete pile.



Simplex pile.

For use in earth that is reasonably firm in its texture and free from water, the preparatory removable pile (see figure 1A)

Fig. 59. Concrete points for simplex piles.

Fig. 60. Simplex pile. Driving pipe form for pile at right. Pulling form and filling at left.

is used. This pile form consists of a length of extra heavy wrought iron pipe, fitted with a suitable driving head of oak, and a conical steel point of a somewhat larger diameter than the pipe, and fitted with an automatic air valve. This preparatory tube is driven into the ground to the required depth, and then withdrawn without difficulty, and the hole so produced is filled with well-

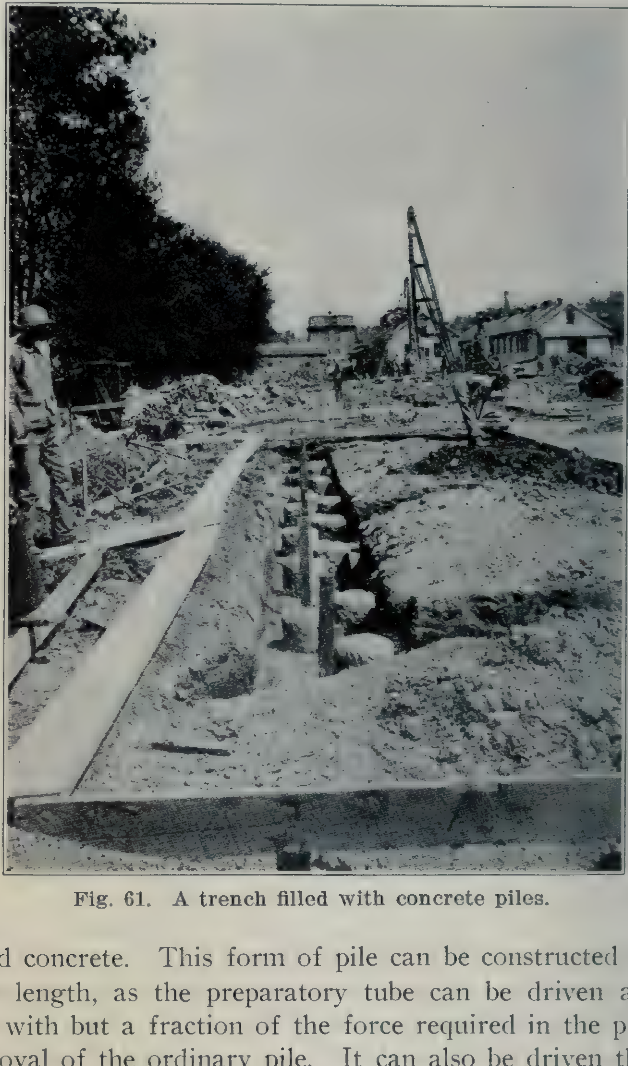


Fig. 61. A trench filled with concrete piles.

rammed concrete. This form of pile can be constructed of any desired length, as the preparatory tube can be driven and removed with but a fraction of the force required in the planting or removal of the ordinary pile. It can also be driven through ground of a density quite impenetrable by any wooden pile and

to almost any desired depth, as there is no appreciable frictional resistance, as the depth increases, either in driving or withdrawing the tube.

The ramming process forces the larger pieces of the aggre-

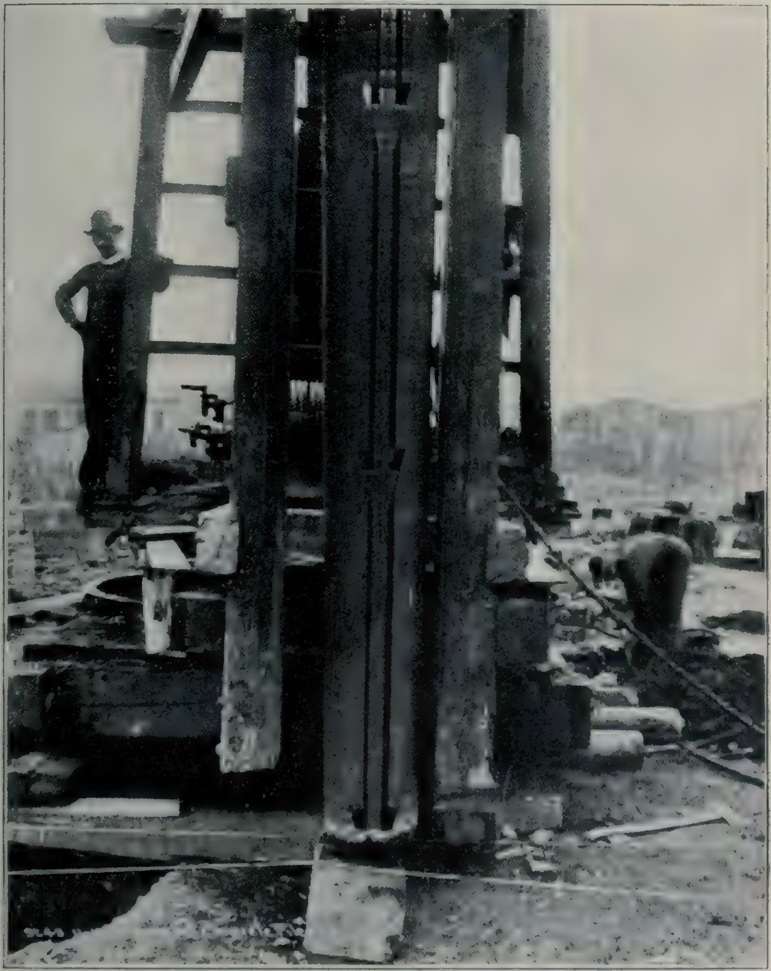


Fig. 62. Raymond Pile Core. This core is used to drive to rock and tapers from 20 to 13 inches. The cut shows core full size as it is driven.

gate into the sides of the hole, materially adding to the frictional hold of the pile on all parts of its surface.

Where the earth is soft, marshy, or where quicksand or

water is encountered, a detachable "point" of concrete (see figure 2A) is substituted for the fixed one of steel. This concrete point is driven to the required depth, and as the pipe is being lifted off, concrete is gradually filled in and rammed home

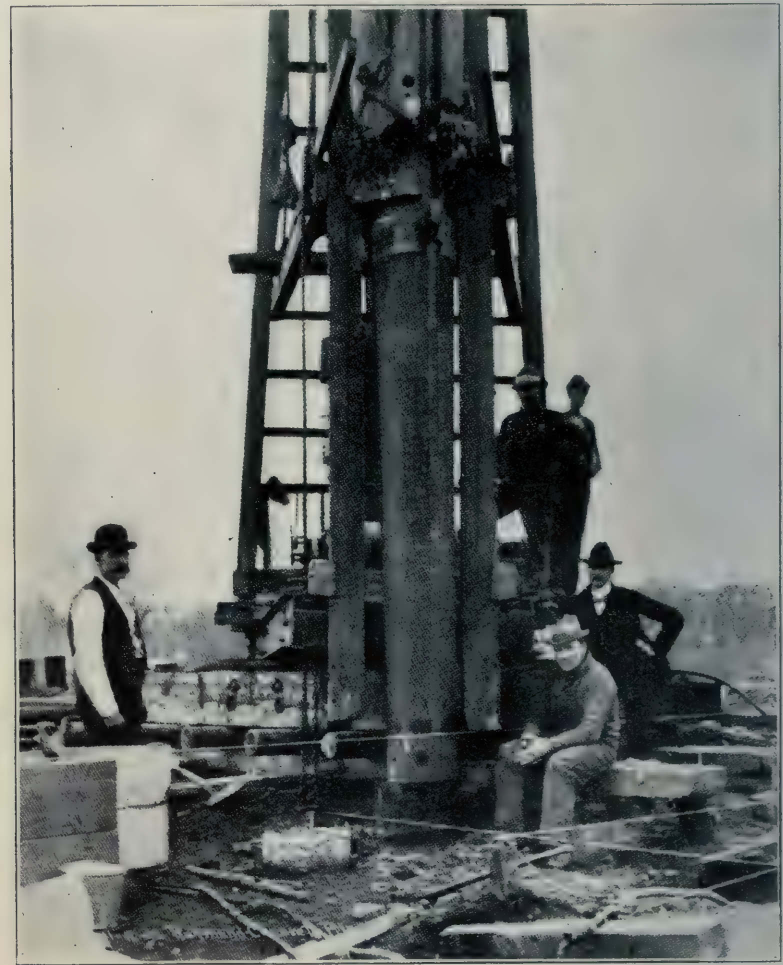
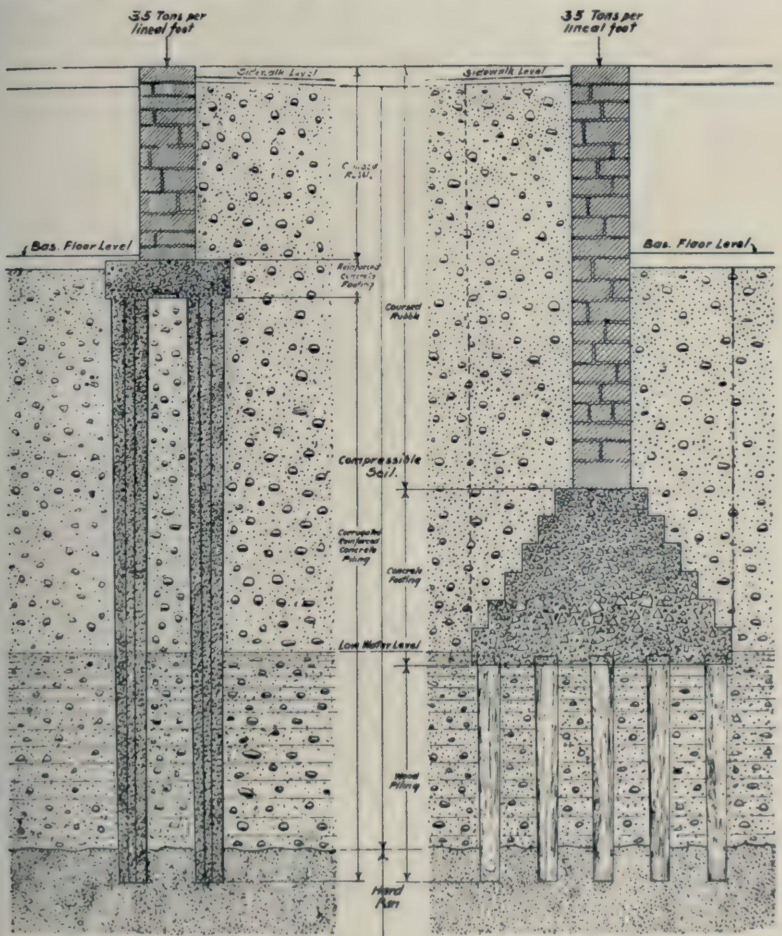


Fig. 63. Pile core collapsed or shrunken, and partly withdrawn from the shell. A completed pile is shown at the left of the shell just driven.

through the pipe, care being taken that a head of the concrete be maintained inside of the pipe while it is being thus gradually withdrawn. By this system all water is displaced and the possibility of the sides of the aperture closing in is entirely removed.

The manner of constructing Raymond concrete piles is as follows: A steel pile-core the size and shape of pile desired is encased in a thin, closely fitting shell. The core and shell are then driven into the ground by means of a pile-driver in the usual manner. By a simple and ingenious device the core is collapsed or shrunken slightly, so that it loses contact with the



Reinforced Concrete Piles. Wood Piles.

Figure 64. Showing the advantage of concrete over wood pile foundations.

shell, and is easily withdrawn, leaving in the ground a clean, perfectly formed hollow tube of the size and depth required, which has only to be filled in with the best Portland cement concrete to complete the pile.

Raymond piles are made of various lengths, tapering generally from 20″ at the top to 6″ at the bottom, making a symmetrical cone affording material resistance to soil penetration by friction.

Concrete piles have this advantage over wood piles; they do not decay, are not subject to destruction by insects and furnish a durable foundation regardless of soil conditions; they can be used in dry filled ground as well as in wet soil, which may dry out and cause wood piles to decay.

A pile foundation generally tends to prevent settlement by the packing and settling of the soil in the vicinity of the pile, frequently rendering the bearing value of the soil around the piles materially greater.

Figure 64 gives a fair idea as to the advantage of a reinforced concrete pile foundation over the wood pile which must be cut off in the vicinity of the low water.

The reader is referred to the catalogues of the Simplex Pile Company, Fifteenth and Chestnut streets, Philadelphia, and the catalogue of the Raymond Concrete Pile Company, Loan and Trust building, Chicago, and also bulletin No. 7 of the Association of the American Portland Cement Manufacturers, regarding the making and driving of concrete piles for more complete information regarding these respective types.

CHAPTER X.

Handling Concrete at Different Temperatures Above and Below Zero.

ARTICLE I.

Concrete Above Zero, Fahr.

Handling concrete to get the best results requires quite different treatment dependent on the temperature. Perhaps the most favorable conditions under which concrete may be placed are temperatures ranging from 45 degrees to 50 degrees Fahr. Under these conditions the concrete does not dry out too rapidly and while it may set slowly it hardens up better than when the temperature is higher.

In hot, dry weather the moisture dries out of the concrete too rapidly, requiring, to secure the best results, that the work be wet down with a hose, particularly during the first day's exposure to the sun after casting. Wetting should commence as soon as the surface of the concrete has set.

Frequently in the hot sun large cracks will open up due to the rapid evaporation of the water. These can and should be promptly filled with a bucket of grout. Sometimes the stone, when exposed to the heat of the sun will become so dried out and hot that it will absorb the water rapidly from the mix and the heat of the stone will be sufficient to set up the concrete before it can be spread in place. Wetting the rock pile down will eliminate this difficulty.

The writer had this experience on one building in the city of Winnipeg, where the stone was piled on the asphalt paving.

At all temperatures below 50 degrees Fahr., it is preferable to warm the water and wake the cement up otherwise it is liable to set too slowly to enable the forms to be safely removed at the usual intervals common in warm weather.

In putting up the Bostwick-Braun building at Toledo, the contractor wired to Minneapolis for the writer to visit the building, stating that the cement in the entire third floor was not

setting up and had been in two weeks. The writer was unable to leave immediately and arrived at the building three days after receiving the telegram. The weather had turned warm in the meantime and the cement had started to set. There were places, however, where a twenty penny nail could be pushed into the concrete with the thumb to its full length. The cement had been mixed with water ice cold, had laid dormant during the chilly weather which succeeded the two weeks after placing the concrete. For the remaining stories, after instructing the foreman to see that the water with which the concrete was mixed was heated to about 120 or 130 degrees there was no difficulty and the forms were promptly removed every ten or twelve days per story although the weather was much colder as the season advanced.

ARTICLE 2.

Concrete Below Zero, Fahr.

In freezing weather it is desirable to wake the cement up by using hot water. Water may be heated to 160 or 180 degrees and the sand and stone mixed with water in the machine before adding the cement. The water will thus warm the stone up and when the cement is dumped into the mixer the temperature will probably be in the neighborhood of about 120 degrees.

At times where the temperature was somewhat below zero the writer has used boiling hot water, handling in this way so that the sand and stone could be warmed up and then adding the cement when the mix would not exceed a temperature of 120 degrees.

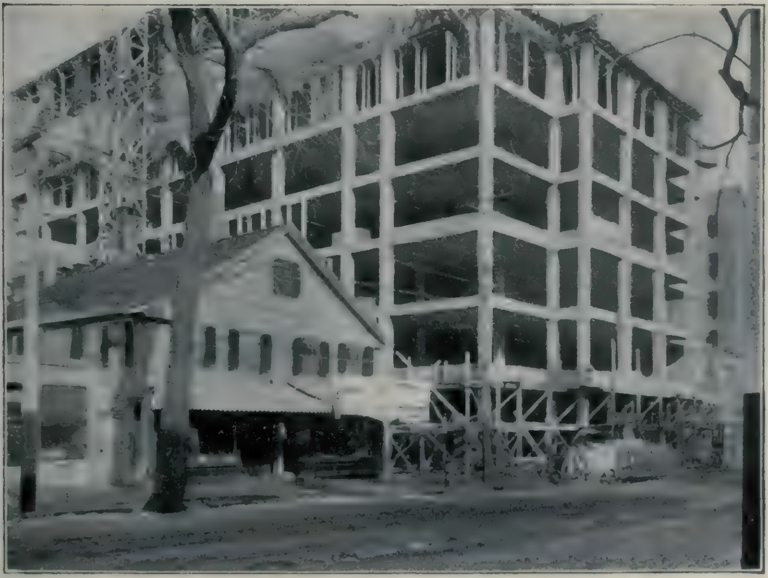
At these low temperatures salt may be advantageously used. The proportion of salt which we find desirable to use for temperatures between 28 above zero and zero is a pint and one-half of salt to each batch containing two bags of cement. Below zero, a little more than a pint per sack and extra care is to be taken in heating the materials and seeing that the concrete gets into place *hot*.

For these very low temperatures it is much better to heat the stone and sand over a coil of steam pipe if such is available. The lowest temperature at which the writer has placed concrete has been 28 below zero and the work in this case was executed in a very satisfactory manner. In addition to the use of hot

water the gravel used as an aggregate was thoroughly heated over a coil of steam pipes. There is much less difficulty in placing concrete in large masses in cold weather than thin slabs and the like.

In the large masses such as thick walls, slabs and the like, the cement generates heat, in setting, sufficiently to keep the body of the material warm, whereas in thin slabs this is not always the case and the concrete may and frequently does freeze.

It comes then to a question as to how to handle the concrete in the best and most practical manner. In a building having



Eight-story mushroom building, Portland, Maine. Erected by the Aberthaw Construction Company, of Boston. Note canvas used around the upper story just cast.

exterior bearing walls the walls are built up then the slab cast and artificial heat should be promptly applied on the under side of the slab to sweat out the concrete and enable it to harden up promptly. Window openings may be readily closed with canvas or light cloth.

Where the building is a skeleton concrete frame it should be protected outside in lieu of the walls by canvas curtain as shown in above figure. Then the concrete may be artificially heated and hardened.

In a large piece of work the most economical method of heating is to put in a small fan with the usual steam coil and heat the building by blowing in heated air in the usual manner. In a small building this is too expensive and the resort is had to salamanders and coke for heating.

ARTICLE 3.

Action of Salt.

The action of salt on concrete is three-fold.

First, it lowers the freezing point and by so doing gives the concrete a better opportunity to attain its initial set before freezing.

Second, it tends to retard the setting and enables us to heat the materials, the cement, water and aggregate to a somewhat higher degree than that we should be able to handle them were it not for the salt in the mix.

Third, as salt has an affinity for water, it retains in the concrete the necessary moisture required for perfect crystallization.

In other words, it prevents the concrete from drying out before it has had time to set when it thaws after freezing.

Calcium chloride has also been used to some extent to prevent the freezing of concrete in cold weather, but owing to the fact that common salt is so much less expensive and readily obtained it is almost universally used by those accustomed to do work in the winter season.

Concrete in setting generates considerable heat after the action of setting has started. In cold weather it requires artificial heating to start this chemical action. An experiment was tried on one piece of work when the temperature was about 25 below zero. A piece of gas pipe was inserted into a column 36″ square, just cast. A thermometer was dropped down the pipe three feet and the upper end sealed with a cork. Upon removing the thermometer eight hours afterwards it registered 95 degrees Fahr.

In placing concrete in a tunnel a considerable increase in the temperature is noted as soon as a considerable mass of the concrete has been placed.

ARTICLE 4.

Curing Concrete Where Proper Precautions Have Not Been Taken.

The engineer is frequently called upon to pass upon concrete which has been placed and the precautions heretofore recommended have not been followed.

The writer has known of cases where the concrete was placed in December, mixed with cold water frozen as fast as placed and this same material thawing out in March was as soft as the day when first cast. Ordering that the concrete be kept thoroughly wet for two and one-half weeks and then allowed to dry out a good hard concrete was secured which after eight months' time stood an exceptionally satisfactory test.

In hardening the material after it has been frozen unless promptly thawed out it sets so slowly that its hardening may be condemned as altogether too slow a proposition for practical purposes if we expect to clean up the work and get it finished within a reasonable time and for this reason it pays the contractor well to heat the materials so that the centers may be removed promptly and the work finished up nearly, if not quite as rapidly as it is ordinarily done in the summer.

The use of salt in the concrete does not appear to impair its strength in the least nor does it appear to have an injurious effect on the metal, provided the metal is well covered with wet concrete.

The use of brine in the mixture is particularly advantageous in placing mass concrete in preventing scaling of the surface from frost action and while it may somewhat retard the setting and hardening the ultimate result appears to be a concrete of even greater strength than that hardened under nominally more favorable conditions of warm weather.

Too great care, however, cannot be exercised in handling work during the winter season since frozen or partly frozen concrete may stand well when the forms are first removed and as soon as it commences to thaw the work will commence to deflect out of shape and look badly if it does not entirely collapse.

The writer has seen many mistakes in judgment in handling work in the cold season of the year, although by the exercise of care and good judgment there is no reason why the work

cannot be executed in a thoroughly first class and satisfactory manner.

In working during the winter season snow and ice frequently get on the forms. This can be readily removed by the use of a steam hose, melting the snow and ice in advance of placing the concrete.

ARTICLE 5.

Precautions in Splicing and Mixing.

In concluding the remarks on handling concrete in cold weather it may not be amiss to call attention to some of the mistakes which the writer has seen made. We have called attention pointedly to the necessity of melting snow and ice on old work and on forms before casting concrete and it remains to call attention to the necessity of special care in the splicing work.

The old concrete may be frozen and not hardened. It will be killed or disintegrated by heating with hot water as some thoughtless foremen have tried to do. Splices in the work should be made with great care and in a vertical plane both for beams and slabs. The old concrete should be cleaned of snow and ice with a steam hose, but no hot water used, then the new concrete may be cast against it and the moderate temperature of the new concrete will gradually soften the old work if frozen and the result will be a satisfactory bond between the two. The writer, however, prefers where practicable to continue casting until a whole floor is complete unless the work is of too great magnitude.

Inclined splices and irregular joints are very decided sources of weakness in work cast in cold weather, in fact, they can hardly be made good unless by digging out some of the concrete and thoroughly grouting the joint after the work has hardened.

The foreman should be cautioned in mixing against killing the cement with boiling hot water. Mixing the sand and stone first with boiling water will take the frost out of the stone and sand and warm it up and reduce the temperature of the water down to 120 or 130 degrees which will not injure the cement. If there is ample salt this temperature may be even ten or fifteen degrees higher for a few minutes and not materially damage the mix.

ARTICLE 6.

Caution Regarding Removal of Forms, Heating Materials, Etc.

A word of caution to the superintendent may not be amiss under this heading. The rapidity of the setting of concrete and hence the time at which it is safe to remove the forms varies materially, dependent on the humidity of the atmosphere. In damp, rainy, wet, chilly weather, concrete is liable to set very slowly indeed. In dry weather and particularly in high altitudes the concrete sets much more rapidly. In Minnesota, for example, with the usual conditions concrete may be counted upon to set more rapidly at temperatures ten or fifteen degrees lower than in situations close to the Great Lakes or in the South where there is a large difference in humidity. These conditions the experienced superintendent soon becomes familiar with for a given locality, but if he moves about it is well to bear these general facts in mind as he will find a marked difference in different sections even with the same cement.

As to the time of removal of the forms the superintendent or foreman should bear in mind clearly the fact that it is not the number of days time that the concrete has been in place or has stood upon the form lumber that determines whether it is safe to remove the forms, but the degree of hardness that has been attained during that period; that concrete may remain on the forms for four months in a northern climate, freeze and thaw out in the spring and be as soft as the day on which it was placed if the foreman has exercised lack of judgment in using cold water to mix the concrete and then allowed the material to freeze after it has been placed.

Frequently as far south as Southern Kansas, damp, chilly weather so retards the setting of the cement not mixed with warm water that after the forms have remained in place a month the construction will not hold its shape, but will sag materially, due to its half-hardened condition. This will never occur where the simple, inexpensive caution has been used of warming up the water at all times where the temperature is below 50 degrees Fahr.

It should be borne in mind by the foreman and superintendent that the slightest sag in the construction, while it may not affect the strength in the least, causes the average owner to be

suspicious of the integrity of the whole work, and as the constructor depends upon satisfied customers in a large measure for future business, and for the prompt payment of the contract price for the work these points should receive careful attention.

The danger of accident with half-hardened concrete is comparatively remote with multiple way systems. As this type of construction will almost invariably give the workman ample time to note its yielding and prop it up before excessive deflection has occurred. Unfortunately, this is not the case with one-way reinforcement. When it once starts yielding as a rule the whole construction goes by the run, and hence from the standpoint of safety to the workmen with this type of construction the superintendent should exercise extreme care.

CHAPTER XI.

Bending Steel.

ARTICLE 1.

Cold Bending with Mild Steel.

In bending rods for beams, columns or slabs, the method used depends somewhat on the character of the steel. With the kind of metal reinforcement recommended, namely, medium steel, nearly all of the work of bending is done cold and at a comparatively insignificant cost. For instance, bending the column rods for the mushroom system on one of the large pieces of work costs about fifty cents per ton. In this case bending machine was arranged, using gears similar to those of the ordinary crab hoist, bending the bar by means of a crank pin on the driven shaft. Bars are not damaged to any considerable extent provided that the radius of the bend is not too sharp and that the moving part bending the bar does not jam the metal so that the flow is confined to a short section.

This is one of the difficulties with quite a number of the lever bending machines which the writer has investigated. He found one used in a piece of work by a contractor on one of his buildings which had actually cracked the bar at the bend. Knowing the metal to be good, tough medium steel, he immediately investigated the bending machine finding that a die with a sharp corner had been used around which to bend the bar. One or two of these bars the writer found had been broken in handling after bending. Fortunately none of them were in a position where direct tensile stress came upon the metal and this die was immediately ordered to be cut over to a reasonable radius of not less than 1½ diameters of the bars.

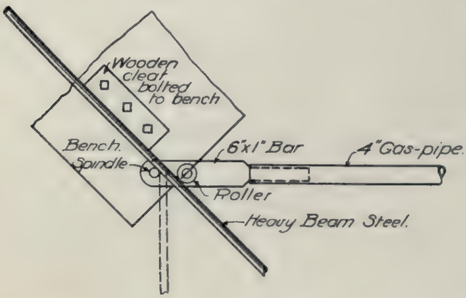
ARTICLE 2.

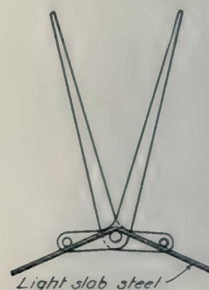
Bending Machines.

Bending light rods, such as ⅜″ and ½″ and the like, may be readily done by the use of tongs or a short piece of pipe

slipped over the end of the rod. Such tongs are illustrated in the accompanying figure.

In general, a good detail for a lever bender is to arrange a small roller on the moving part of the bending lever so that the pressure is brought against the bar by a roll and that there





may then be no tendency to localize the stretch of the metal at one place by friction, thereby seriously injuring the bar.

For ring rods and the like, such as are used in the mushroom system, an ordinary set of blacksmith's tire rolls is the most convenient equipment for this class of bending.

For spiral the same set of rolls is frequently used.

The accompanying three figures show one of the most convenient bending machines the writer has seen, invented by Theodore Kordong, of Minneapolis, Minn.

Fig. A shows the bending of a mushroom column rod, the

Fig. A.

stop on the circular segment fixing the angle of bend.

Fig. B shows the same machine bending a hook on the end of a rod, while Fig. C shows a beam rod being bent, having several bends.

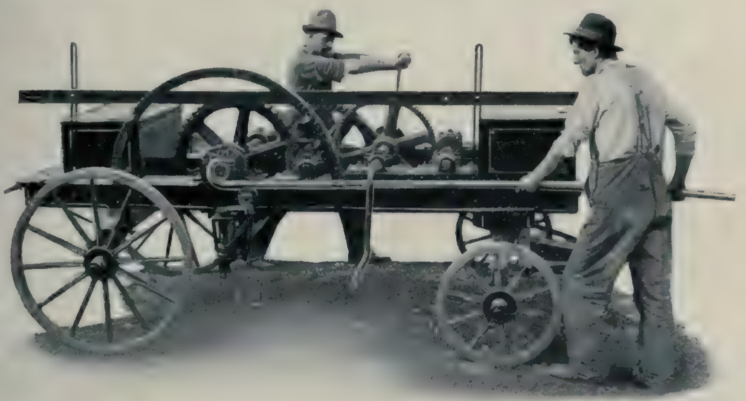


Fig. B.

ARTICLE 3.

Hot Bending and Precautions with High Carbon Steel.

In bending bars hot, which is frequently done where they are over 1″ in diameter, there is frequently carelessness in overheating the steel. Warming up to a low cherry red is the highest heat which should be permitted by the foreman in charge of such work.

High carbon steel is more readily injured by overheating

Fig. C

than mild steel. It is too hard to be worked cold and can only be bent to the desired form by heating. In heating it is much more liable to be severely damaged than mild steel and hence extra care should be taken when using this grade of metal, to see that it is not burned by the smith.

In bending cold twisted bars, where specified by the architect, they should be invariably heated, otherwise in endeavoring to bend them cold the damage done the bar by the cold twisting will manifest itself in brittleness of the bar and the tendency to crack or break at the point where the bend is being made.

CHAPTER XII.

ARTICLE 1.

Protection of Steel and Iron from Corrosion by Portland Cement.

Deterioration of steel by corrosion or rusting is one of the difficult problems in nearly all structures intended to be permanent.

Paint of linseed oil combined with some pigment is used for the protection of structural steel and its efficiency depends on the complete removal of rust before painting. Further, this coat of paint must be renewed at frequent intervals.

Fortunately in concrete steel construction we have in the cement the most perfect protective coating known for iron and steel. Bars somewhat rusty if placed in wet concrete and removed after one week will be found perfectly clean, the rust having been chemically destroyed by the cement.

The bond between cement and steel is formed better between bars somewhat rusty when placed in the concrete than between bars new from the mill. The reason seems to be that a small amount of rusting removes the black mill scale and allows the cement to come in contact with the solid bar. Paint, oil or grease tend to weaken the bond between the two materials.

Bars, removed from cement after over twenty years' exposure of the specimens to the elements, have been found bright and as good as when first placed in the work.

This protection however is dependent entirely on the thorough covering of the steel by the wet concrete and hence the importance of using a plastic mixture—one that will flow slowly and thoroughly surround the steel, requiring only puddling rather than tamping to secure substantial work.

It may be noted incidentally also that exactly the same kind of mixture is essential if we are to secure smooth, neat looking work. That is, work without ragged patches showing rough stone with the voids not filled with mortar.

S. B. Newberry states the chemical theory of protection of iron embedded in concrete as follows:

"The rusting of iron consists of oxidation of the metal to the condition of hydrated oxide. It does not take place at

ordinary temperatures, in dry air or in moist air free from carbonic oxide. The combined action of moisture and carbonic acid is necessary. Ferrous carbonate is first formed; this is at once oxidized to ferric oxide and the liberated carbon dioxide acts on a fresh portion of metal. Once started the corrosion proceeds rapidly, perhaps on account of the galvanic action between the oxide and the metal. Water holding carbonic acid in solution soon, if free from oxygen, acts as an acid and rapidly attacks iron. In lime water or soda solution the metal remains bright. The action of cement in preventing rust is now apparent. Portland cement contains about sixty-three per cent lime. By the action of water it is converted into a crystalline mass of hydrated calcium silicate and calcium hydrate. In the hardening it rapidly absorbs carbonic acid and becomes coated on the surface with a film of carbonate, cement mortar thus acting as an efficient protector of iron and captures and imprisons every carbonic acid molecule that threatens to attack the metal. The action is, therefore, not due to the exclusion of the air, and even though the concrete be porous, and not in contact with the metal at all points, it will still filter out and neutralize the acid and prevent its corrosive effect."

ARTICLE 2.

Protection and Provision for Plumbing.

In plumbing fixtures a considerable amount of lead piping is used. This should be either entirely eliminated where it comes in contact with concrete or well protected by a heavy coating of tar or asphalt paint as the author has seen pieces of lead piping removed from concrete which has been transformed almost completely to red oxide.

Cast iron, wrought iron or steel and brass fittings are not injuriously affected.

In general, provision should be made for all large fixture pipes as digging large holes in the concrete should not be encouraged since frequently these come at points which may seriously weaken the construction.

For example, the writer has known of cases where the plumber thoughtlessly dug a hole right through the center of a beam leaving an insignificant amount of concrete each side of the hole to take care of the shear and cutting one of the main beam rods, in so doing thus forcing the slab to carry the load to a great extent.

ARTICLE 3.

Placing Electric Conduits, Gas Pipes, Etc.

The most convenient position in reinforced concrete work for gas pipes, conduits and the like, is to bury them in the middle of the slab with outlets at desired points. So buried, conduits if of moderate size, in no wise weaken the construction. They should not, however, be placed beneath the reinforcement. This is a mistake that is too often made.

A conduit, say ¾″ pipe, is placed right along on top of the centering with perhaps ⅛″ of concrete under it in the finished work. Reinforcing bars rest on top of the conduit pipe and probably dip downward in the slab on each side of the conduit to a greater or less extent. Then, as soon as the centering is struck and the strain comes upon the rods there is a tendency to straighten out under pull, to cause the slab to deflect or sag and open up large unsightly cracks near the bottom of the conduit pipe. The reduction in strength due to this position of the pipe may be as much as ten or twelve per cent of the strength of the slab. In the ceiling the crack, from the standpoint of appearance is unsightly and leads to somewhat unwarranted suspicion of extreme weakness.

The standard outlet boxes as usually furnished by electric supply companies are unfortunately much too shallow. They should be deep enough so that the pipe connections can be readily made without interference with the reinforcement. The writer has frequently had wood plugs turned up and put in these boxes in order to keep them up at the proper elevation and give an opportunity to place the conduits without bending and kinking them as they enter the box.

Provision for openings in floors for steam pipes, soil pipes, leaders and the like, may most economically be made by placing thimbles of sheet metal (filled with sand) on the forms in the desired location thus saving the expense of cutting later.

When holes have to be cut through the slabs the cutting should commence from the bottom. If the hole is cut through from the top as soon as the drill or chisel strikes the bars a large unsightly chunk will be broken out of the under side of the slab. As it is quite difficult to patch these places with plaster the architect should not allow the work to be done in this manner.

ARTICLE 4.

Plastering on Reinforced Concrete Work.

This is a feature of concrete building construction which is of considerable interest to the architect. It is decidedly annoying for a client to come to the architect and state to him that a large section of the plaster has dropped off from certain sections of his building. This happens far more frequently than the advocate of reinforced concrete likes to admit, although when the causes of the failure of plaster to adhere to the work are fully investigated and the work then properly executed there is little trouble from this cause.

In general, the average plasterer is in the habit of plastering on wire lath, wood lath or the like. With such a base upon which to work there is ample opportunity for a lean mortar to clinch in a firm and satisfactory manner. When plastering on concrete, however, plaster is held in position by adhesion only to the concrete. There is little or no chance for efficient clinch or mechanical bond as is the case when plastering on lath or wire cloth. The materials, the concrete and the plaster, do not have exactly the same coefficient of expansion and they are held together by adhesion between the two, and evidently this will be greater the richer the plaster mortar. It will be greater as the surface of the forms used for centering are rough sawed lumber rather than surfaced lumber. The tendency to drop off will be less the thinner the plaster coat and the less damage can result from the fall of any given section of plaster; hence, a thin coat of plaster is to be preferred to a thick scratch coat and finish coat thereon.

Lime mortar well gauged with Portland cement just before use will adhere better to reinforced concrete than the Gypsum or patent plasters. Any plaster will adhere to concrete best when the concrete is thoroughly set and dry. Trouble almost invariably results from the attempt to plaster on concrete before it has had a chance to thoroughly dry out and set hard as it seems that the moisture from the concrete prevents the plaster from drying and setting properly.

Washing the surface of the concrete before plastering with a solution of one part vinegar to three parts clean water greatly improves the bond between the two materials, as it removes the inert matter from the surface of the concrete.

Some plasterers prefer to coat the concrete work with R. I. W.

or other tar paint in advance of applying the plaster to secure a more satisfactory bond.

The writer has seen considerable trouble with plaster upon reinforced concrete, though in all cases, on investigation, he has found either the plaster was applied upon partially cured concrete or improperly put on.

Sometimes the plasterer will endeavor to put on a thick coat, get air bubbles between the new plaster and the concrete and these expanding and contracting with each change of temperature will gradually loosen up quite a large area of the plaster coat and after six or eight weeks it will drop off in large chunks.

The remedy for this difficulty as the writer views it is as follows:

First, proper attention to see that a rich mortar is used.

Second, to see that the concrete work is thoroughly dried before attempting to plaster it.

Third, to make the finish as thin as possible. A skin coat 1-16 or 1-32 thick being ample to make a good finish.

Fourth, to thoroughly wash the under side of the surface of the slab with the vinegar solution recommended.

Fifth, to see that in centering the floor that the rough side of the lumber is used next to the concrete, giving a rough surface rather than a smooth surface for the plaster to stick to.

Sixth, to avoid the use of soap, grease or benzine to prevent the concrete from adhering to the centering.

Nearly all of the patent gypsum plasters will, when applied wet to steel or iron, badly corrode the metal. Fortunately this corrosion seems to continue only until the plaster sets but it is sufficient to stain badly the plaster in the vicinity of the metal. It may be prevented in the manner recommended for the protection of lead in concrete.

CHAPTER XIII.

ARTICLE 1.

Fireproof Properties of Concrete and the Protection of Steel from Heat.

The value of concrete as a fireproof material has been pretty well demonstrated in a large number of severe conflagrations and also in many fire tests by the building departments of various cities. In fact, it may be stated that concrete ranks as the best fireproof building material and it is this quality together with its low cost which is responsible for the enormous increase in its use.

Intense heat injures the surface of the concrete, but it is so good a non-conductor that if sufficiently thick it provides ample protection for the steel reinforcement and the interior of the mass is unaffected even in unusually severe fires.

For efficient fire protection in slabs under ordinary conditions with one-way reinforcement the lower surface of the steel rods should be ¾″ above the bottom of the slab. With two-way reinforcement this may be reduced to ½″ as should one layer of rods be overheated the upper layer is still amply protected.

Structural beams, girders and columns should have at least 2½″ of good concrete for efficient protection. Beams having large rods the thickness of the concrete coating outside of the rods should never be less than 1¼″ nor less than the diameter of the largest rod used in the beam. In columns the outside area should be considered as fire protection and no dependence placed upon it in figuring the strength of the section, in carrying the working load.

These limitations are sufficient for ordinary purposes. Where, for example, a factory building is to be put up where there will be scarcely any inflammable materials to be stored, it is a waste of money to provide a thick massive concrete simply to resist fire. On the other hand, where the building is to be used for storage of material capable of creating not

only a hot fire but an intense heat of long duration, special provision may be made by using an excessive thickness of concrete for fire protection though in such a situation a sprinkler system should be preferably employed.

One of the most severe practical tests occurred in a fire at the Pacific Coast Borax Refinery at Bayonne, N. J. This building was a four-story factory, built entirely of reinforced concrete except the roof. The contents of the building, the roof and interior wood trim were destroyed, but the walls and floor remained intact except where an eighteen-ton tank fell through the roof and cracked some of the floor beams. The heat was so intense that brass and iron castings were melted to junk. A small annex built of structural steel frame was completely wrecked and the metal bent and twisted into a tangled mass.

In general, the fire resistance of Portland cement concrete is governed or affected by the character of the aggregate and the amount of cement in the mortar. In other words, we may state the general rule that the richer the mortar or the greater the amount of cement used the greater the fire resisting properties of the concrete. Second, as regards the aggregate, the smaller the stone the better the fire resisting qualities.

Trap rock will make a concrete presenting greater resistance to extreme heat than lime stone or granite.

The writer has been advised by parties interested in a smelter in the Globe district that their superintendent succeeded in substituting for fire brick a mortar of one part Portland cement to one and one-half pure silica sand in furnace flue lining and found that it gave better satisfaction than the fire brick.

Rich mortar makes a stronger concrete better able to resist severe temperature stresses while the high proportion of cement when dehydrated on the exposed surface makes a very perfect non-conductor. The damaged portion acts as a coating of non-conducting material, preventing the uninjured parts from further or rapidly progressive injury.

ARTICLE 2.

The Theory of Fire Protection.

The theory of fire protection is given by Mr. Newberry as follows:

"Two principal sources from which cement concrete derives

its capacity to resist fire and prevent transference of the heat to the steel are its combined water and porosity. Portland cement takes up in hardening a variable amount of water, depending on surrounding conditions. In a dense briquette of neat cement the combined water may reach twelve per cent. A mixture of cement with three parts sand will take up water to the amount of about eighteen per cent of the cement contained. This water is chemically combined, and not given off at the boiling point. On heating, a part of the water goes off at about five hundred degrees Fhr., but the dehydration is not complete until nine hundred degrees Fhr., is reached. This vaporization of water absorbs heat and keeps the mass for a long time at a comparatively low temperature. A steel beam or column embedded in concrete is thus cooled by the volatilization of water in the surrounding cement. The principle is the same as in the use of crystallized alum in the casings of fireproof safes; natural hydraulic cement is largely used in safes for the same purpose.

The porosity of concrete also offers great resistance to the passage of heat. Air is a poor conductor, and it is well known that an air space is a most efficient protection against conduction. Porous substances, such as asbestos, mineral wool, etc., are always used as heat insulating material. For the same reason cinder concrete, being highly porous, is a much better non-conductor than a dense concrete made of sand and gravel or stone, and has the added advantage of lightness. In a fire the outside of the concrete may reach a high temperature but the heat only slowly and imperfectly penetrates the mass, and reaches the steel so gradually that it is carried off by the metal as fast as it is supplied."

In regard to cinder concrete the writer would add that, first, it is not a desirable material to use from the standpoint of strength. Second, that as usually employed, insufficient cement is used to make a good fire resisting material. Thus Prof. Norton compares the action of stone and cinder concrete in the Baltimore fire as follows.

"Little difference in the action of the fire on stone and cinder concrete could be noted and as I have earlier pointed out the burning of bits of coal in poor cinder concrete is evenly balanced by the splitting of stone in the stone concrete. I have never been able to see that in the long run either stood fire

better or worse than the other. However, owing to its density, the stone concrete takes longer to heat through."

Perhaps if the relative proportion of cement were the same in each, the cinder concrete, if the cinders are real clinker, would prove the better fire resisting material as Mr. Newberry assumes. This point cannot be too well emphasized.

A concrete must be rich in cement to make a first class fireproof material and for this reason alone a leaner mixture than 1-2-4 should not be allowed in an important building.

Thus far our attention has been primarily directed to the fireproof qualities of concrete as such. In considering the fire resisting properties of the composite material known as concrete

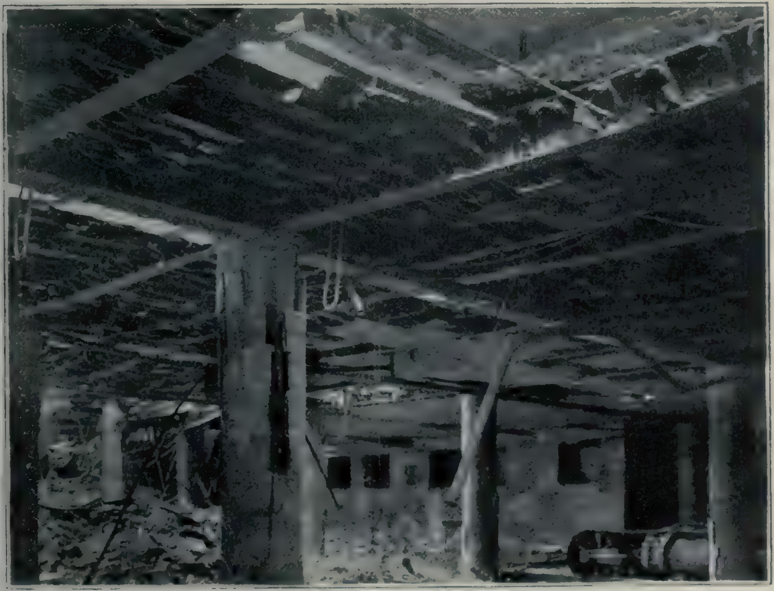


Fig. 65. Showing effect of fire on tile construction.

steel or reinforced concrete we must consider the effect of the unequal heating of different parts of the construction. We have previously noted that the coefficient of expansion of steel and concrete are practically identical. Their coefficients of heat capacity and conductivity, however, differ widely and for this reason the distribution of the metal in the form of small bars rather than in large units will give a more satisfactory result from the fireproof standpoint.

The difficulty with the combination of tile and structural steel or terra cotta as a fire resisting material lies largely in the fact that the coefficient of expansion of the two materials is radically different.

This is well illustrated in figure 65, showing the effect of heat, breaking and cracking tile between steel beams after exposure to a severe fire.

ARTICLE 3.

Concrete vs. Terra Cotta.

Prof. Norton in his report on the Baltimore fire to the Insurance Engineering Experiment Station, states:*

"Where concrete floor arches and concrete steel construction received the full force of the fire it appears to have stood well, distinctly better than the terra cotta. The reasons I believe are these: First, because the concrete and steel expand at sensibly the same rate, and hence when heated do not subject one another to stress, but terra cotta usually expands about twice as fast with increase in temperature as steel, and hence the partitions and floor arches soon become too large to be contained by the steel members which under ordinary temperature properly enclose them. Under this condition the partition must buckle and the segmental arches must lift and break the bonds, crushing at the same time the lower surface member of the tiles.

"When brick or terra cotta are heated no chemical action occurs, but when concrete is carried up to about 1,000 degrees Fahr., its surface becomes decomposed, dehydration occurs, and water is driven off. This process takes a relatively great amount of heat. It would take about as much heat to drive the water out of this outer quarter inch of the concrete partition as it would to raise that quarter inch to 1,000 degrees Fahr. Now, a second action begins. After dehydration the concrete is much improved as a non-conductor and yet through this layer of non-conducting material must pass all the heat to dehydrate and raise the temperature of the layers below, a process which cannot proceed with great speed."

In the composite material of concrete and steel in the form of a continuous concrete monolith there are severe temperature

* Eng. News, June 2nd, 1904.

stresses set up by the unequal heating of different parts of a floor during a fire and the manner in which the material will withstand these stresses will depend in a large measure on how thoroughly the steel is disseminated through the concrete enabling it to take up the tensile stresses induced by this unequal expansion in the various parts caused by the unequal heating, hence that type of construction which is reinforced practically in all directions is best calculated to withstand the severe stresses so originated. Further, as the concrete is injured or disintegrated on its surface the smaller the surface exposed the less will be the damage, and the fewer irregularities in the form of the construction, the less it will be injured.

Looking at the question from this standpoint then, type four (see page 19), or the flat slab type of construction would rank first from the fireproof point and type three second. In other words, the natural concrete types which are in no wise imitations of older types of construction are far better adapted to resist the severe conditions of a conflagration than those types which are merely an imitation of older forms of construction.

In reporting to the Chief of Engineers, U. S. A., regarding one of the reinforced concrete buildings which passed through the Baltimore fire, Capt. Sewell writes:*

"It was surrounded by non-fireproof buildings, and was subjected to an extremely severe test, probably involving as high temperature as any that existed anywhere. The concrete was made with broken granite as an aggregate. The arches of the roof and the ceiling of the upper story were cracked along the crown, but in my judgment very slight repairs would have restored any strength lost here. Cutting out a small section—say an inch wide—and caulking it full of good strong cement mortar would have sufficed. The exposed corners of columns and girders were cracked and spalled, showing a tendency to round off to a curve of about three inches radius. In the upper stories, where the heat was intense, the concrete was calcined to a depth of from ¼ to ¾ inch, but it showed no tendency to spall, except at exposed corners. On wide, flat surfaces, the calcined material was not more than ¼ inch thick, and showed no disposition to come off. In the lower stories, the concrete was absolutely unimpaired, though the contents of the building

* Eng. News, March 24, 1904.

were all burned out. In my judgment, the entire concrete structure could have been repaired for not over twenty to twenty-five per cent of its original cost. On March 10th I witnessed a loading test of this structure. One bay of the second floor, with a beam in the center, was loaded with nearly three hundred pounds per square foot superimposed, without a sign of distress, and with a deflection not exceeding ¼ inch. The floor was designed for a total working load of 150 pounds per square foot. The sections next to the front and rear walls were cantilevers, and one of these was loaded with 150 pounds per square foot, superimposed, without any sign of distress, or undue deflection."

In concluding the subject of the fireproof qualities of concrete it may be well to call attention to the stock argument of the burned clay advocate.

A small specimen of burned clay or terra cotta if subjected to a temperature of 2,000 degrees and then immersed in water will remain undamaged.

A small sample of concrete subjected to similar treatment will be totally disintegrated. Hence the burned clay advocate argues concrete is not a suitable fireproof material.

The fallacy in this plausible argument as has been pointed out in an excellent editorial in the Engineering News lies in the fact that the conditions in a building during a fire and in the furnace are radically different.

In a fire in a building the concrete is not exposed to heat on all sides, nor is it exposed for any long continued time to very high temperatures. The greatest heat is generally near the ceiling when the surface, as noted in Capt. Sewall's report, may be dehydrated slightly and protect the material back of the injured portion. The net result is that less damage results than is done to the terra cotta or hollow tile, since the latter does not expand in unison with the supporting steel frame, and is crushed and broken by the severe temperature stresses resulting from this cause.

Combination structures of hollow tile and concrete are open to the same criticism from the fireproof standpoint, namely, the combination of two elements in a composite structure having radically different coefficients of expansion. Evidently the expectation that the combination will, under severe conditions, prove satisfactory cannot be realized.

ARTICLE 4.

Rates of Insurance on Concrete Buildings and Contents.

Boards of fire underwriters representing the older line companies, have been somewhat slow in recognizing concrete as a fireproof material and it seems to the concrete constructor frequently that they do not recognize the great difference that there is in this material dependent on the character of the mixture and dissemination of the metal reinforcement.

The position that some of these boards have taken in rating the mill building with a sprinkler system lower than a concrete building without a sprinkler is a position hard to explain except possibly that members of these boards are financially interested in sprinkler system equipments.

On the other hand, the mutual companies appear to have been more progressive and are writing policies at rates that appeal to the constructor as far more consistent and rational.

Comparing the lowest rate which has come to the writer's attention for a timber building, mill construction, used for mercantile purposes, equipped with sprinkler system, A. D. T. watchman service, etc., with the lowest rate which has come under his notice for a reinforced concrete building similarly equipped with a sprinkler system, the rate for the concrete building was less than one-half that for the timber building, being a six-cent rate for the concrete structure against a twelve and one-half-cent rate for the timber building. The advantages from the fireproof standpoint may be stated as follows:

(1) A well designed reinforced concrete building offers security against disastrous fire and total loss of structure.

(2) It reduces the danger of damage to the contents by preventing the spread of fire from floor to floor.

(3) It prevents damage to the contents by water from story to story.

(4) It renders sprinklers unnecessary in buildings whose contents are not especially inflammable.

(5) It reduces the danger of panic and loss of life incident thereto among employes or occupants of the building.

Evidently in order to prevent the spread of fire from floor to floor, the floors should be continuous, or openings properly protected by automatic shutters or doors. Thus, if we are to protect the goods or contents on the floors above from fire below,

it is necessary to have the elevator shafts protected by automatic fire doors and stairways cut off in a similar manner. This can be done at a comparatively small expense.

Protection from exterior exposure may be readily made by the employment of wire glass, metal frames and the like, in place of wood frames and ordinary glass windows.

A good concrete floor is practically waterproof and a slight pitch with suitable scuppers would practically eliminate water loss in floors below from flooding a floor in which fire has broken out in the contents or goods stored thereon.

In the ordinary factory or mercantile building with wood floors, loss from water is frequently greater than the loss by actual fire where an incipient blaze has been extinguished.

In the concrete building, on the other hand, each floor becomes almost a waterproof roof. Frequently a tenant moves into the lower stories of a concrete building before the upper portion is complete, the floors above acting as a roof.

According to Mr. Kunhardt, vice-president of the Boston Manufacturers Mutual Fire Insurance Company, these mutual companies take a business-like stand regarding the extent of fire protection required in each individual case. While the value of the automatic sprinkler is recognized and the general rule specifies its installation the Factory Mutual Companies do not require it in the concrete building except where there is sufficient inflammable material in the contents to furnish fuel for a fire.

An essential feature in good factory construction includes not only consideration of the building but protection adequate to its needs only. The extent to which the above is faithfully carried out will eventually be the determining feature in the cost of insurance.

Mr. Kunhardt gives the following table:

	All Concrete		Brick Mill Construction or Open Joists.		Wood Mill Construction or Open Joists.		Add for Brick or Wood Buildings in Small Towns and Cities Without Best of Water and Fire Departments
	Bldg.	Cont's	Bldg.	Cont's	Bldg.	Cont's	
General Storehouse	20c	45c	60c	100c	100c	125c	25c
Wool Storehouse	20c	35c	40c	60c	75c	100c	25c
Office Building	15c	30c	35c	50c	100c	125c	25c
Cotton Factory	40c	100c	100c	200c	200c	300c	50c
Tannery	20c	40c	75c	100c	100c	100c	25c
Shoe Factory	25c	80c	75c	100c	150c	200c	50c
Wollen Mill	30c	80c	75c	100c	160c	200c	50c
Machine Shop	15c	25c	50c	50c	100c	100c	25c
General Merchandise Bldg.	35c	75c	50c	100c	100c	150c	25c

These costs are based on the absence of automatic sprinklers and other private fire protective appliances of the usual completely equipped building. They are not schedule rates, but may be an approximation to actual costs under favorable conditions based on examples in various parts of the country.

As illustrating the value of fire protection, Mr. Kunhardt states, that in the Boston Manufacturers' Mutual Companies, the average cost of insurance on the better class of protected factories has now for some years averaged, excluding interest, less than seven cents on each hundred dollars of risk taken, and on first class warehouses connected with them, one-half of this amount. These figures can be compared with the table as illustrating the gain by the installation of proper safeguards for preventing and extinguishing fire.

In these same protected factories and warehouses the *actual fire and water loss* is less than four cents on each $100 of insurance and he regards it possible to reduce this loss materially, practically along the lines we have outlined.

ARTICLE 5.

Some Literature on So-called Fireproof Types of Buildings.

The report on the fire in the Parker building, by the New York Board of Fire Underwriters, published on April 22, 1908, is an extremely valuable paper, thoroughly discussing the faulty characteristics in the fireproofing of this structure, a large sec-

tion of which collapsed. The frame was of cast iron columns, heavy steel beams and tile arches.

An amusing feature in the newspaper literature on this disaster was that the collapse of this building was immediately misrepresented as a failure of reinforced concrete, when there was not a particle of reinforced concrete in the floors, and merely a weak cinder concrete filling over the top of the 8″ terra cotta flat arches.

The burned clay interests have been extremely industrious in knocking reinforced concrete through the press of the country and prejudicing to the greatest extent the general public. In fact, to read some of the papers especially devoted to the burned clay interests one would think that concrete was only suitable for footings or some place under ground where it could not supplant brick, tile or other clay products.

In the earthquake at San Francisco the weakness of the hollow tile is well brought out in the numerous reports from that city. Among these reports is that by Mr. Himmelwright, although written admittedly by an exponent of a concrete system and perhaps with some prejudice towards the type of construction he favors, certainly gives a large amount of valuable information and numerous cuts of photographs of the havoc wrought by the earthquake and fire.

Under earthquake conditions it seems evident that no type of construction is as well suited to stand the jar and quake as properly constructed natural types of reinforced concrete building, for the reason that even where brick bearing walls are used if the slab following good practice is cast on top of the brick work covers the wall except the outside face brick and becomes a part thereof, the building is tied together in a manner far superior to that which it is possible to attain with any other type of construction.

The last point named is an important one. A structural steel skeleton which merely has the steel beams anchored in the walls and then encased in concrete having a supporting floor slabs which only abut the walls or at best is only cast on a small corbel, does little towards tying the construction together in a satisfactory manner, and such a building could not be expected to stand the racket as well as one of the natural concrete types noted.

CHAPTER XIV.

Floor Finish, Stair Details and Roof Insulation.

ARTICLE 1.

Strip Fill.

Proper time for the application of the strip fill is immediately after the rough slab is sufficiently hardened to work upon for the reason that at this time the strips can be readily spiked down to the practically hardened concrete and wedged up or lined up to the proper levels without difficulty, then the strip fill can be put in with the same rig that is used to cast the floor slab.

For strip filling as noted in an earlier chapter the writer prefers to make the mixture approximately the same as that for the slab except where the loads are light and strength is no object. Then a one, three and one-half, four mix or even five is ample for all purposes. No natural cement or lime should be used in the mixture as where it is used trouble almost invariably results due to its slow hardening; moisture of the strip filling not thoroughly dry swelling and expanding the flooring to such an extent that it springs away from the fastenings to the strips necessitating entire relaying of the floors in many cases.

ARTICLE 2.

Strips.

Strips are conveniently made by splitting up old centering lumber as 4x4's, ripping them through the center and then ripping the 2x4's with a bevel cut giving a strip 1½ on top, 2¼ on the bottom and 1¾ deep. This is a good way to work off the old material.

ARTICLE 3.

Widths of Flooring.

Narrow widths of maple flooring are preferable to wider widths. Two to two and one-half inch widths are as wide as the

writer cares to recommend if the floor is $\frac{7}{8}$; where $1\frac{1}{4}$ or $1\frac{3}{4}$ thickness is used the writer would prefer to limit the width to three or three and a quarter inches.

ARTICLE 4.

Cement Finished Floors.

In no part of concrete construction work has there been so much difficulty to get a first class and satisfactory piece of work as in putting on concrete finished floors.

The difficulties to be met with are as follows. First, a good bond is desired between the finish coat and the concrete of the rough slab.

Second, if this finish coat is cast immediately following the casting of the rough slab in order to get a better bond in that way the finish must be protected first against being cut and marred by working on it and centering the next higher floor. In addition thereto the casting of the concrete columns for the next floor will cause unequal moisture conditions in the finish coat at and around the base of the column to those in the center of the slab resulting invariably and regardless of the system employed in shrinkage cracks around and near the base of the column. Those ignorant of the strength of the construction frequently consider the shrinkage cracks of this character as an actual indication of the weakness.

In several cases contractors putting up the mushroom system have put on the finished floor before erecting the next story with the inevitable result of shrinkage checks around and near the foot of the column. These slight checks have been largely advertised by certain competitors as weak points in the system, a rather amusing statement when we stop to consider that this is the point of maximum strength and that at this point, which they would like to have the public consider weak, there is at least four times as much reinforcement in the concrete proper as in any other part of the construction; the absurdity of the claim becomes apparent on its face.

The next difficulty that is encountered is from the idea that it requires a wet mixture to secure a good bond to the old concrete. When a wet mix is used it is leveled off and the workmen are obliged to wait until the cement is partially set when they pro-

ceed to trowel it over and smooth off the finished surface. In doing this some of the cement at the surface has already attained its final set while some has not progressed that far. The net result is that a portion of the cement which has attained its final set combined with some inert material brought to the surface by troweling forms a dust which is readily rubbed up on the finished floor. The condition of the floor is better or worse finished in this wise dependent on the following conditions:

Where temperature conditions are such that the cement hardens very slowly, as in the fall of the year, and the finish is allowed to stand four or five hours before it gets sufficiently hard to work upon, the resulting finish is most inferior. Where, however, the temperature conditions are such that the cement sets much more rapidly, much better surface conditions are secured, sometimes one that is fairly satisfactory. As the writer looks at it a better way to obviate this difficulty is as follows:

Apply the finish coat after the removal of the forms and after the rough floor slab has been cleaned and brushed off with a steel brush. Then wet it down thoroughly and allow the water to soak in for some hours, then wet it again and grout the surface with a neat cement grout and apply the finish coat mixed comparatively dry. Not so dry but what it is plastic enough to hold its form if a hand full of the material is squeezed together in the hand but not sufficiently wet to flow. This material should immediately be rammed in place until the water is brought to the surface, then the workmen can be put to work and immediately trowel down the finished surface before the cement has had time to take its initial set. As soon as the floor has stood a few hours and set up somewhat it is then policy to thoroughly wet it down and properly cure the concrete. In this manner the writer believes that much of the trouble with concrete finished floors can be eliminated.

ARTICLE 5.

Treatment of Floors.

A concrete floor may be treated in a manner somewhat similar to a wood floor. It may be varnished and painted if desired. Where a floor has been put down and the finish is unsatis-

factory from the standpoint of dust, if not too bad, the trouble may be remedied by the application of boiled linseed oil or other preparations. Where, however, the surface is unusually bad there is no remedy except by rubbing it down in a manner similar to that by which the surface finish is secured on terrazzo floors.

ARTICLE 6.

Mixture for Finish Coat.

A good specification for finish coat is one cement to one and one-half clean coarse sand. A small amount of thoroughly hydrated lime is sometimes used to advantage. Many architects specify granite. This is a good aggregate but unfortunately the feldspar dust seems to work to the surface and where the sand is of the right character the writer decidedly prefers it to crushed granite.

ARTICLE 7.

Concrete Stairs.

Reinforced concrete provides an inexpensive means for building stairways which are far more nearly fireproof than any other type which we can construct.

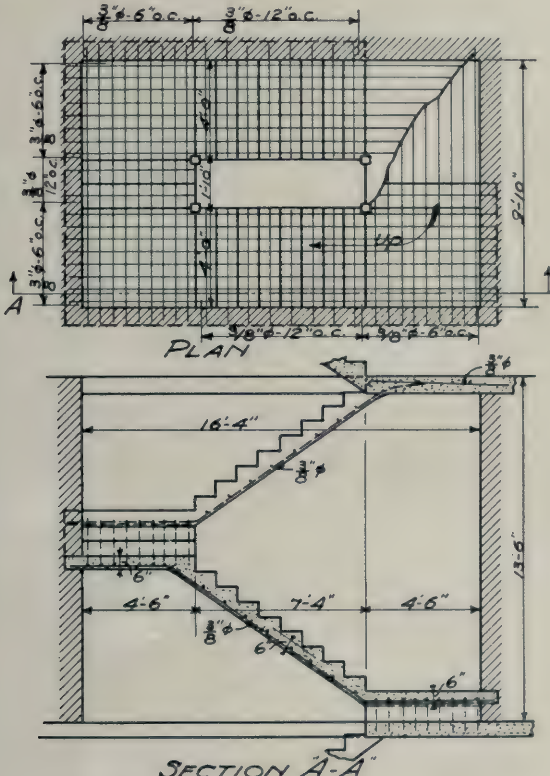
The accompanying figure, page 191, shows the usual method of reinforcing used.

For ordinary heights ⅜″ rounds, 6″ centers are ample for the inclined slab. The inclined slab is generally built 4½ or 5″ for ordinary runs and the horses are cast on top of the incline. Where fancy treads are desired they are sometimes made of the White Portland cement with crushed quartz. This makes a very durable tread and a material which is next to marble and will wear somewhat better.

Frequently marble or slate treads are used and these can be readily bedded on the concrete horse and the riser brushed up, rubbed and painted or varnished as preferred. Frequently it is desirable to suspend stair platforms without supports from below.

This can very readily be done by dropping the slab rods down to the level of the platform from the slab above on one or more sides and encasing the suspended rods in an ordinary 2″ partition of cement plaster for fire protection. Fastenings

for metal hand rails can readily be cast in the end or top of the stairs as the work is placed.



Detail Stair Reinforcement.

ARTICLE 8.

Insulation.

Those not familiar with reinforced concrete frequently make the mistake of designing roof slabs in a cold climate without insulation. The result is that the moisture in the warm air in the room below the roof slab is condensed on the under side of the cold slab and continually drips at all temperatures, which keep the slab colder than the air within the room. This is readily remedied by a cinder filling from four to six inches thick. In fact, we frequently recommend to our clients that instead of putting on a roof slab proper that the ceiling slab be cast level

which might at some future date be used for a floor should a story be added and on this slab to build up with cinders sufficiently to give the standard pitch-and-gravel roof the usual slope. This slope should preferably be in the neighborhood of 5-16 to 3/8" to a foot of run. On the top of the cinder insulation we customarily put in a 1" coat of cement mortar mixed about one cement to three sand which forms a good base for the pitch-and-gravel roof. In place of the cinder filling sometimes a false roof is built up using the old centering lumber. Where this is done all openings through the ceiling and roof should be encased or protected by a concrete fire wall, then no further damage can occur than the burning up of the comparatively inexpensive false roof should the same catch fire from above.

The ceiling slab, should the roof burn away, would protect the goods or business carried on beneath it until the false roof could be replaced.

Insulation is of the utmost importance in concrete roofs in the climate of the United States, north of southern or central Kansas. The writer would hesitate to allow any work to be executed with his guarantee against this difficulty in latitudes north of this. Perhaps on the Pacific Coast insulation might be omitted nearly as far north as Portland without serious difficulty resulting.

CHAPTER XV.

ARTICLE I.

Commercial Conditions Affecting the Safe and Rapid Introduction of Concrete Steel Construction.

Any new industry concerning which people are not posted, is almost invariably a rich field for the ingenious and enterprising shark. Concrete construction has proved no exception to the rule.

Thus we have the theorist, who has never put up a building, writing a treatise on the subject. This theorist likely secured his limited stock of information in a manner similar to that of the dude reporter detailed to write up a piece of work. This reporter came to the job, wearing kid gloves and patent leather shoes. And so that he might not get his tailor-made suit spattered with mud, but might contemplate the work at a safe and respectful distance, he thoughtfully provided himself with opera glasses.

Is it any wonder that serious accidents occasionally result when some poor innocent "Book-taught Bilkins" follows some of the highly scientific theories so originated and developed?

Then we have the imitation or would-be architect who neither knows nor appreciates the breadth and dignity of the profession to which he pretends to belong. This "Pin-Headed" specimen thinks that if he can make a bluff at knowing it all he can build an enduring reputation. So, after investing two dollars and fifty cents in some publication on reinforced concrete, he immediately becomes a full-fledged expert on the subject and proceeds to put up a building which quite likely tumbles down—too frequently on the wrong man's head.

Meeting a salesman for some kind of a "No-burn-system," the writer was shown a little nickel plated model of a combination structural reinforcement for concrete columns and was asked how it looked to him.

"Looks don't count." "What it is worth in the combination is what I am interested in," was the reply.

"You are an engineer and ought to know the price of steel," the salesman replied.

"True, as to the price per pound but what it will develop in point of strength in the combination is the question." "What tests have been made?" "How do you figure the combination from them?"

"O, those are engineering questions and you know you do not have to know anything about engineering to sell steel to an architect." "My time is too valuable to be bothered with such insignificant details." "I can hire a cheap engineer to make tests for me." "Why, I have made eighty thousand dollars this past year on a deformed bar which I invented."

The writer gravely congratulated this brilliant salesman on his commercial success but frankly told him that if he knew more about the business he thought he would be more successful in selling steel.

This is only one of the many examples of the totally irresponsible men selling material to go into reinforced concrete work that comes under the head of reinforcement. Such men assume absolutely no responsibility for the finished product and would prefer to make a sale of one-half enough metal for a building and take chances on collecting their money than to worry or bother about the insignificant detail as to whether the building would be safe or a menace to the lives of those who put it up or occupy it.

As little is known regarding the exact mathematical distribution of shear through this composite material we hear a great deal about it from the salesman who knows the least concerning reinforced concrete. In fact, the term, *provision for shear*, is the keynote of the "Con" song by which the oleaginous salesman frequently hypnotizes the unsophisticated purchaser into buying a type of reinforcement for a price one hundred or one hundred and fifty per cent above its intrinsic value as a reinforcing metal. Thus he offers a member in which there is ample provision for shear at the center of the beam where the shear is supposed to be zero and little or insignificant provision for shear at the end where the shear is known to be a maximum.

Then, we have each of these brilliant classes of salesmen running down his competitor with one tale of woe or another invented for the occasion, as to the horrible failures which have occurred under the other fellow's system. They make lengthy

remarks on the fact that this or that multiple way system is not scientific because it cannot be figured—although it safely carries the load and notwithstanding the fact that a multiple way system has never been known to let go and injure those putting it up.

If these men are selling metal at double its value in the combination of concrete and steel, how do they do it my critic may ask?

The answer is easy. The man who is selling metal at double its value can readily cover this little discrepancy if he is dealing with a contractor lacking in familiarity with the cost of the work. If he can persuade the contractor into the belief that he can execute centering at half its cost and handle concrete on a small job at the cost it was handled on a large piece of work with ample equipment, he has made a sale of steel and the contractor has bit off a job on which he is bound to lose money.

But, the critic will object, how can they continue to do business along this line? The old saying among grain men that a new sucker is born every minute has apparently held true in the introduction of reinforced concrete. While the same man will not get burned twice a new man steps up at once ready to pay the price for a similar experience.

The preceding is a brief outline of the actual conditions one meets in the commercial field of reinforced concrete work.

As the material, its properties, and the cost of executing work, become better understood we may look for a decided change in the general methods of doing business.

The fact that there have been so few failures under the conditions above outlined speaks volumes for the general honesty, integrity and reliability of by far the great majority of the contracting firms throughout the country.

ARTICLE 2.

In dealings with some building departments, commissioners and inspectors throughout the country, the writer has been not a little reminded of the story of the old woman and her sailor son. On a visit home the young man, thinking to interest her, told her about the flying fish. This she could not believe, but when the graceless scamp told her about the time they had anchored in the Red Sea, and on heaving up their anchor found

a section of one of Pharoah's chariot wheels hooked onto the fluke, she knew it must be so because the "good book" told how the wicked king was drowned with his mighty hosts in this same Red Sea. So some building commissioners, if shown a multiple way system of reinforcement which is economical, safe to erect and figured upon proper economic lines, do not believe that it will carry the load although the same thing has been tested hundreds of times elsewhere. Yet if shown a really dangerous type of simple, independent beam construction, worked in upon a structural steel frame work without ties or any of the true monolithic features of reinforced concrete proper, they immediately accept it without questions because there is some good book on reinforced concrete which tells how it should be figured.

ARTICLE 3.

Constitutionality of Building Ordinances.

A word regarding the constitutionality of building ordinances may not be amiss.

The object of regulations governing the construction of buildings is primarily to protect the public from the unscrupulous owner or contractor who might be tempted to put up a type of construction which is unsafe.

A regulation which bars a type which can be shown to have an ample margin of safety defeats the primary object of the ordinance and becomes on its face unconstitutional.

Too much attention is paid to the mere letter of the law by some scrupulous but narrow gauge engineers occupying the position of engineer of the department of buildings. He is open to criticism by competing companies if he passes on a new or novel type in any manner which can be construed as in any wise favoring such construction by recognizing its developed strength if the strength so developed cannot be figured in accordance with some formula embodied in the letter of the law which was in no wise designed to cover the particular type under consideration.

It may be questioned whether such interpretation can be considered justifiable from the legal standpoint or that of good engineering ethics. But the fact remains that the owner is frequently called upon to pay a large additional price due to the fact that the building ordinances of too many of our cities are

framed, as regards concrete construction, by men not familiar with it and who do not know how to draw up rules and regulations which will promote the advance in safety and economy of this excellent type of non-combustible building construction.

In actual practice the result of not a few of our city building ordinances is to perpetuate the construction of antiquated types which are in reality little better than fire traps and are a menace to public safety. Such construction annually involves an enormous property loss through fire and results too frequently in such horrible disasters as that at Collinwood, Ohio, where a large number of children were burned to death in one of that class of buildings which are unduly encouraged by building ordinances through lack of knowledge exhibited in almost complete failure to recognize fundamental principles of economy in the design of reinforced concrete.

The building ordinances of comparatively few cities contain that sensible provision to be found in the regulations of the bureau of inspection in regard to the use of reinforced concrete in the city of Philadelphia, which reads as follows:

The Chief of the Bureau of Inspection may from time to time issue such modifications to these regulations as may be found necessary to conform to modern practice.

ARTICLE 4.

The Term Factor of Safety.

A popular misconception regarding the meaning of the term factor of safety as applied to steel construction is another point which has exerted an adverse influence on the rapid introduction of concrete construction from the economic standpoint.

Many have the mistaken idea that a factor of safety of four in steel construction means that the construction may be loaded to four times the rated working capacity. This is not the case, as the yield point of the metal is only about twice the working load; hence the actual factor of safety is about two against the nominal factor of four.

In other words, the nominal factor of safety of four in structural steel work is based on the ultimate carrying strength in tension of the metal which is about four times the working load, but after the load has reached about double the working load we have nearly or quite reached the yield point value of

the steel and it commences to stretch, pulling out sometimes as much as twenty per cent or more, for mild steel, its total length before breaking. Evidently when this plastic distortion commences in a beam or column the member is soon so deformed that we cannot figure its strength in the frame thus limiting the ultimate strength to practically a little more than twice the working load for this nominal factor of four.

In properly designed concrete construction the concrete is made stronger than the steel for the reason that it is generally economical so to do, and hence the strength of the steel is the strength of the concrete construction, and we should not consider it reasonable to expect to test the steel more in the case of the concrete construction than we dare do in structural work, and hence double the working load is a fair test for this class of work, and in reality, in view of the fact that the cement improves with age, if it will stand this test at the age of from three to four months the owner can rest assured that the factor of safety is greater than with structural steel construction.

ARTICLE 5.

General Principles Governing the Responsibility for the Safe Construction of Reinforced Concrete.

Having given a brief outline as to the commercial conditions which militate against the safe and rapid introduction of reinforced concrete we will attempt a short discussion as to the relative safety of general types together with the accountability of those connected with the work.

(1) The responsibility of engineer or designer.
(2) The responsibility of the contractor.
(3) A safeguard for the owner.

ARTICLE 6.

Responsibility of the Engineer.

The engineer is accountable for the selection of a type of design which is safe to erect. That is, a design in which a sudden collapse cannot readily occur. He should so design his work that it can be executed by the exercise of ordinary care. He should design it so that there shall be a minimum chance of bad work through lack of care on the part of the workmen.

We have called attention to the fact that concrete is a material naturally best fitted for monolithic construction, that the natural concrete types are best tied together reinforcing the construction to act as a continuous monolith. To do this ample lap of the bars is essential over all supports whether bearings or supporting columns.

An example of a failure due to the lack of lap of the bars over the support is well illustrated in the collapse of the Bixby Hotel at Long Beach, California.

In the design of this building the bars reinforcing the beams were of insufficient length if placed exactly symmetrical about the center of the span to lap over the column more than about two inches. The probabilities are that the bars were not placed symmetrically about the center and that there was little or no lap at the point of initial failure.

That this failure started in an upper story where the small diameter of the column gave little or insignificant lap and that the lower stories, where the larger columns provided greater lap, were erected without mishap, emphasizes the point made.

Where there is insufficient lap owing to the shrinkage of the concrete in setting we have not only the shear on partly hardened concrete but also the tensile shrinkage stresses tending to crack the concrete through at the point where the bars are not sufficiently lapped over the supports.

The engineer or designer is responsible for failure to provide a type of column design in which there are no obstructions in the shaft of the column to preclude securing a solid casting. Failure of the building of the Eastman Kodak Company, of Rochester, New York, appears to have been due in a large part to prongs on the column reinforcing bars projecting into the column and interfering with the placing of the concrete in a manner to secure solid castings.

In a case of an eleven-story building, the architect used eight vertical reinforcing bars and tied them across the shaft of the column with numerous ¼″ ties. In pouring the concrete in several columns these ties blocked the flow of the concrete and when the columns were apparently full it was found on removal of the forms that there was a large void two or three feet in height in several of these columns due to the interference of the ties in pouring the column. The designer has no excuse for the employment of such details.

The construction should be preferably monolithic. Continuous beams should be used rather than simple beams. Failure of the Luxfer Prism Company's building in Chicago is of a character that should emphasize this point.

A slab reinforced in two directions and supported on four sides may be loaded until it is cracked through and if the slab is a large one may be loaded until the deflection is twelve or fifteen inches and still carry the load which broke the construction down at this point and strained the steel beyond the yield point value.

A slab reinforced in one direction only will on the other hand break down completely and sometimes let go quickly and almost without warning. This is especially true where forms are prematurely removed.

We have noted in a previous chapter under "Principles of Economic Design," that the true concrete types which are continuous monolithic construction have the lowest coefficient of bending, hence there is little excuse on the part of the designer for failure to adopt the safest type of construction particularly in view of the fact that it may be figured with greater certainty and a higher degree of scientific accuracy than the so-called simple types of simple beams or one way reinforcements that have been used in this composite type of construction.

While the engineer may be held responsible for accurate computation and for those features bearing upon the safety of the design, he cannot prevent, unless on the ground, the inexperienced foreman from knocking centers at too early a period. He cannot prevent the deflection of reinforced concrete work where the material has not been allowed sufficient time to properly harden before the removal of the forms. If, however, his design is one of the two natural concrete types of construction there is little danger of a sudden collapse and the worst that can happen will probably be the necessity of digging out some work which has got out of shape due to lack of judgment and haste on the part of the erection foreman or superintendent.

ARTICLE 7.

Responsibility of the Contractor.

The contractor is primarily responsible:

(1) For honest execution of the work.

(2) The use of sufficient cement.

(3) Securing the proper aggregate.

(4) Erection of centering of proper strength and staying it so that collapse will not occur during erection.

(5) For exercise of care in leaving forms in until concrete has properly set.

He should be responsible for simple tests on cement as recommended under cement. He should be responsible for seeing that splicings are properly made between new and old work. That the reinforcement is placed as required by the engineer's plans, and while he cannot be held responsible for the design he should exercise greater care in putting up the less conservative types which consist of one way reinforcement than is essential in putting up multiple way systems or natural concrete types.

As regards aggregate, the contractor is responsible for the use of reasonably clean, coarse sand and hard stone.

ARTICLE 8.

Removal of the Forms.

The question as to the proper time for removing centering is one which unfortunately cannot be answered in terms of so many days, but only in terms of the degree of hardness that the concrete has attained during the period in which the forms have been left in place.

Cement sets slowly or rapidly, dependent on its activity and more largely on the temperature under which it hardens. In the fall of the year if the concrete is mixed with water ice cold and it remains chilly sometimes the cement will not start to set for ten or twelve days and then if it becomes warm it will harden up quickly.

Hardening may be hastened by warming the water and waking the cement up. Treatment in freezing weather will be handled in a separate chapter.

The question of determining the hardness of the concrete and as to whether it is safe to remove the forms is one which the contractor or foreman must decide. As a rough test the concrete should be sufficiently hard so that a twenty penny nail will double over and cannot be driven into it more than three-quarters of an inch. A good idea can be obtained as to its hardness by trying it with a hammer and seeing how readily it can be in-

dented, as well as by driving a nail into it and finding out its condition under the surface. Examining the concrete around openings, etc., will enable the experienced foreman to form a correct judgment as to whether it is safe to remove the centering. In any case these rought tests are sufficient to determine that the removal of the forms is a safe proposition for the workmen.

In long span slabs or beams there may be some sag due to the compression of the concrete if it has not set sufficiently hard although no accident results. Such deflection or sagging tends to destroy the owner's confidence in the work although it may have no material effect from the standpoint of strength. In fact, where a long span slab or a beam has sagged a moderate amount before the concrete is thoroughly hard there is generally little loss of strength.

In the case of a slab, if it is afterwards leveled up with additional concrete, it is stronger than that portion of the work which has kept its shape. The owner considers this an evidence of weakness. The good natured contractor knows that if he has filled up the depression in a panel which has sagged slightly it is probably the strongest slab in the building and is quite willing to make an exceptionally fine test at this point.

The time which the centering should remain in place varies for different spans. With a slab sixteen or seventeen feet, under favorable drying conditions, it should be possible to remove the forms in eight or ten days. Where the span is longer, say twenty or twenty-five feet, two weeks at least should be allowed. unless the slab is extra thick. For example, a slab eight inches in thickness and twenty-five feet in span should be allowed ample time to thoroughly harden if it is to keep its shape immediately after removal of the forms. Whereas a span of the same length eleven inches thick would only need practically the same time as the shorter span on account of the additional thickness. These are practical points which it is well to bear in mind as upon them commercial success in a measure depends.

ARTICLE 9.

A Safeguard for the Owner.

The owner should be held accountable for the selection of an experienced contractor or experienced superintendent to look after his interests in the work. He can readily provide if he

would, *a safeguard against the stinted use of cement by furnishing himself at his own expense the cement to be used in the work,* leaving the contractor no opportunity to gain by the reduction of this necessary element in the combination.

He should as a matter of personal interest carefully look up the record of the engineer or superintendent whom he places in charge of his work.

While there is no safer type of building to erect than a well designed concrete steel structure honestly executed, a poorly designed building involves a risk to workmen almost if not quite equal to that incident to the erection of structural steel.

CHAPTER XVI.

ARTICLE 1.

Rapidity With Which Concrete Steel Construction May be Erected.

No type of building can be as rapidly or quickly designed, detailed and erected, as the natural types of reinforced concrete construction. If we take type four, for instance, a single computation is sufficient for a panel and where panels are tabulated for various loads two hours' work is sufficient to make the computations for a given size factory building or manufacturing plant, including an estimate of the cost of reinforcement, quantities of concrete and centering with sufficient precision for bidding purposes.

In no type of building construction can the materials be secured as promptly as for the reinforced concrete structure. An ordinary four or five-story building can sometimes be erected complete in the average time required to get the shop details out for a structural steel frame. Especially where the building is irregular in form there is this advantage with reinforced concrete, that the joints are made with the cement in plastic form, that the rods can be lapped more or less over the supports, eliminating the necessity of a large amount of figuring required for the skew connections of structural work, hence the engineer's end in this line of building construction is greatly simplified.

Actual examples of the average speed with which this type of construction is carried on by the well organized contracting firm is sufficient to render this statement clear.

ARTICLE 2.

Illustrated Examples in the Warm or Favorable Season.

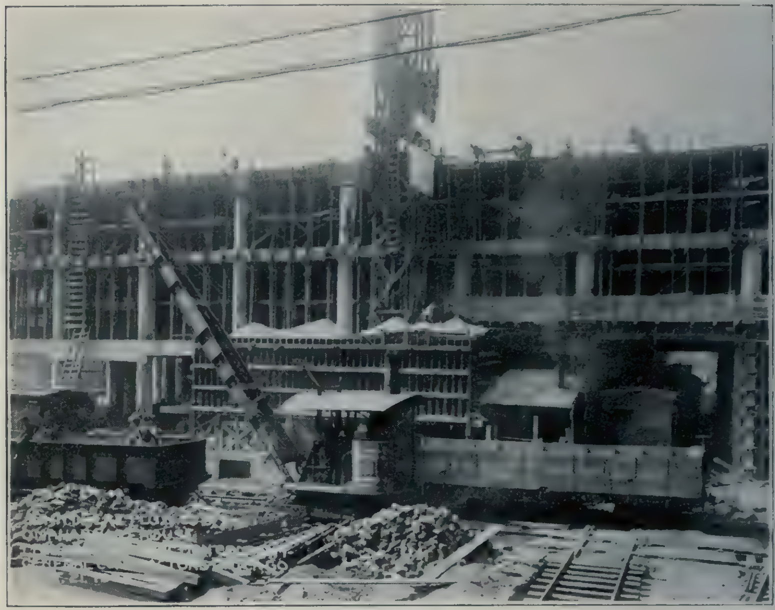
We illustrate herewith a series of views of the John Deere Plow Company's building of Omaha, giving the dates on which the photographs were taken. This is a typical warehouse building, 135x285′. From the photographs it will be noticed that

Mushroom System.
John Deere Plow Co. Building, Omaha.

View of first floor reinforcement and part of second floor centering. Condition of work Aug. 17, 1908.

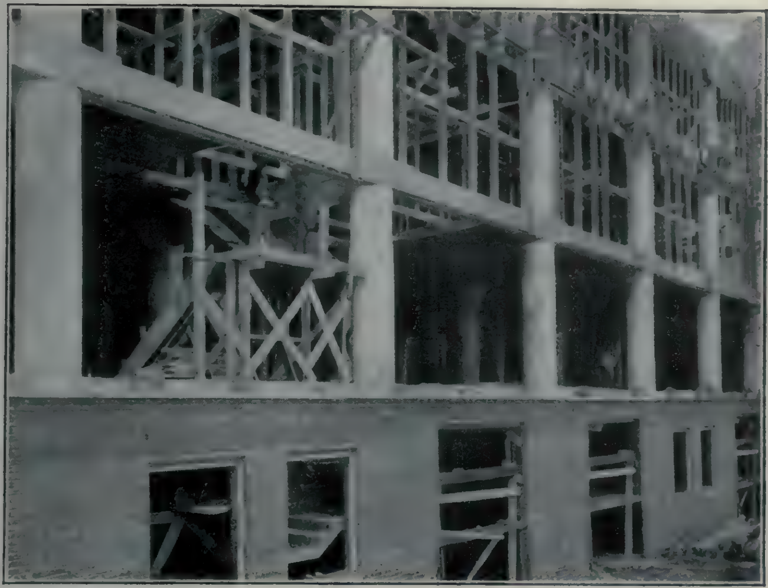
Condition of work Aug. 31, 1908.

John Deere Plow Co. Building, Omaha.

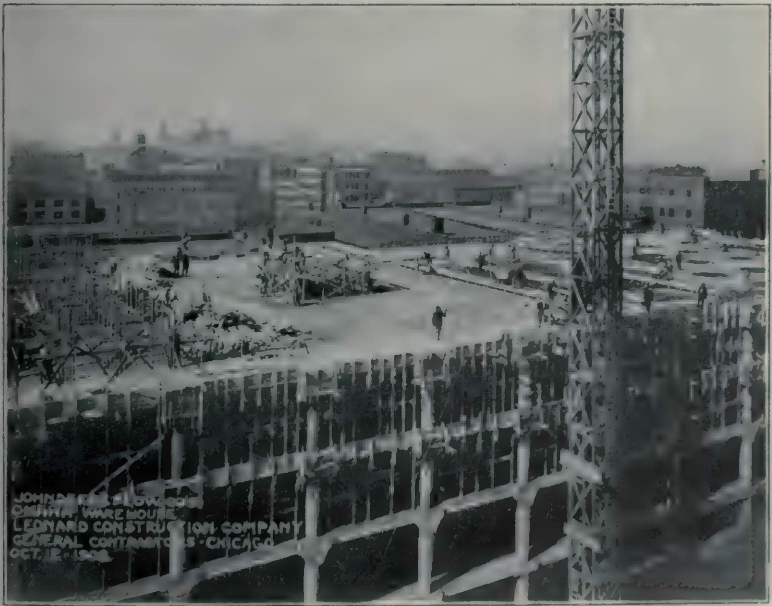


Condition of work Sept. 21.

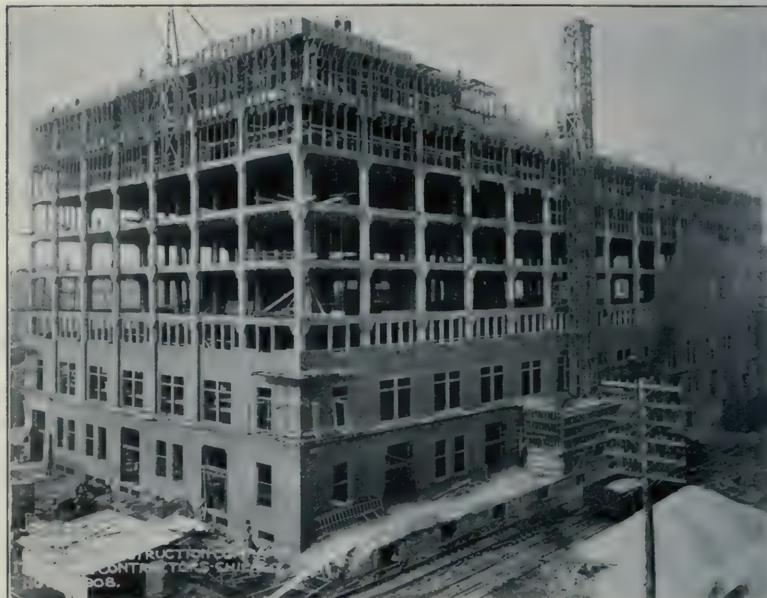
View of second floor after removal of forms. Note weight of material on first floor. Taken about Oct. 1.



Detail view of wall girders of John Deere Plow Co.'s Building, Omaha, Nebraska. Taken about Oct. 1, 1908.



View taken Oct. 12. Working on sixth floor and putting up centering for the seventh.



View taken Nov. 2, 1908.

Side view, under construction, showing handling of materials by locomotive crane. Concrete work erected by Leonard Construction Co., of Chicago, at rate of one story per week. View taken Nov. 9, 1908.

Exterior of Building. View taken Dec. 14, 1908.

Test of Building. Note this test not made until Feb. 13, 1909.

Erected in Ninety Days.

Wisconsin Central Freight Station, Minneapolis, Minn. Side elevation, under construction, showing concrete skeleton.

C. N. Kalk, chief engineer W. C. Railway, C. A. P. Turner, consulting engineer. Butler Bros., contractors.

the average progress was about one story per week or about 5,500 square feet of floor per day including erecting the column forms and slab forms, placing the steel and casting the concrete. This work was executed by the Leonard Construction Company of Chicago; C. A. P. Turner, designing engineer; O. A. Eckerman, architect; Fisher & Lawrie, consulting architects; A. N. Talbot, engineer for owner.

In St. Paul, Butler Bros. succeeded in putting up about a story every ten days of good working weather on the Lindeke-Warner building. This work consisted of erecting exterior brick carrying walls, slabs and columns and the strip fill ready to receive the finished floor. This building was 235 by 165, and this rate of progress was about at the rate of 4,000 square feet per day, an even better record than that made at Omaha, when we consider the additional work done.

A. Bentley & Sons, of Toledo, Ohio, made an equally good record on the Bostwick-Braun building of that city. This building was 220 feet square on the street fronts, eight stories high.

Bostwick-Braun Building.

August 1st, showing seawall and adjacent footings incomplete.

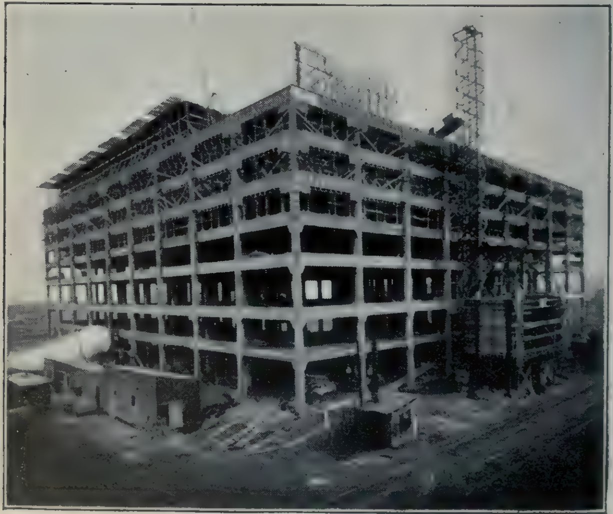
Nov. 11th, showing centering nearly complete for the roof, eight stories in place.

View from water front.

Bostwick-Braun Wholesale Hardware House, Toledo, Ohio.

This is the largest wholesale hardware building in existence, constructed throughout with a concrete skeleton and curtain walls. It is 220 feet square on the street fronts and somewhat trapezoidal in plan, due to the direction of the river front.

The concrete work in this building was put up at the remarkably rapid rate of one floor per week after the foundations were in place. George S. Mills, architect. C. A. P. Turner, engineer.



View showing Summit street front.

View of the second floor Lindeke-Warner building, St. Paul, Minn., under construction. Building 235x165 feet. Story every ten days. (See page 132 for elevation of completed building.)

ARTICLE 3.

An Example of Rapid Erection in the Winter Season.

The buildings heretofore noted were erected during the summer or favorable season for reinforced concrete work.

As an example of the rapidity of construction during the winter season, Butler Bros. Company, of St. Paul, Minn., put up the eleven-story Booth Packing Company's plant in that city in ninety days in the middle of a Minnesota winter.

It would be hard to find these records excelled in any other type of building construction.

ARTICLE 4.

Relative Difference Between the Large and the Small Job.

As regards the difference between the large and small building, it is fair to note that the time per story is about the same almost regardless of the size of the building, since with the larger building it is possible to rig up in a manner that will enable the handling of the work more rapidly and the fact that there is the large area enables the employment of more men and makes it possible to keep at work on the various features such as centering, pouring concrete, placing steel, in one continuous operation. The centering going ahead for one floor, the steel being placed following up the carpenters and the concrete men in turn following up those placing the steel and when the carpenters are through with this floor they immediately proceed to erect the forms for the next on that portion of the floor where the concrete has been cast.

Where the amount of work to be handled runs eight to ten thousand yards it pays well to rig up with overhead bins and mixing plant and to arrange for the use of half-yard dump cars in placing the concrete. Where the yardage is much less the wheelbarrow or two-wheeled truck and scale hoist becomes an economic method of handling material. The later mixers are arranged with a charging device which saves wheeling material up an incline as was customarily done in earlier work.

CHAPTER XVII.

Centering.

ARTICLE 1.

Materials for Wood Centering.

Centering is one of the largest elements entering into the cost of reinforced concrete work. Figure p. 216 shows the centering used in the Minneapolis Knitting Company building, a structure which we have termed type III. The joists used were 2x6, 20-inch centers, with 1″x6″ fencing for the floor. For studding 4x4 are usually used, spaced about seven feet apart, capped by 2x8 double and resting on wedges by means of which the centering can readily be adjusted to the desired level of the finished floor.

For square columns of small section 2x4s spiked together, forming the square ties, are about as cheap as any method of putting the boxes together.

For columns some use 4x4 side pieces, slotted at the end and ½″ bolts. This allows the same frame to be adjusted for different size columns and makes a very substantial form, but somewhat expensive in first cost. For beam boxes 1⅜″ plank for bottom and ⅞″ boards for sides are preferable. For plain slab forms the following is the writer's preference, where lumber is used:

Joist 2″x8″, 20 to 22″ centers, 1″x6″ fencing for sheathing, 2x8s double for ledgers spaced eight to nine feet apart. Vertical posts seven to eight feet centers. The 4x4 verticals butted under the ledger pieces and the ledger prevented from turning on top of the 4x4 by short pieces of ⅞x4, nailed to both ledger and top of 4x4 with 5d nails. The bottom of the posts are best adjusted by wedges 12″ long, cut from 4x4s. This will allow the leveling up of the centering very readily.

In centering shown on page 216 the column boxes are 1½″ stock banded by 2″x4″ lapped and fastened together with wire spikes. Beam boxes were made up of ⅞ boards and 2x4s forming vertical frame and 1x6 bottom of same. A light ledger is nailed along the side of the beam box to receive the joist for

View showing centering in process of erection.

supporting the slab. The beam box was then braced up and two lines of supports placed under the 2x6 joist.

Sometimes it is desirable to center, using a size of material which can be worked into boxing as is used for hardware storage purposes, implement house requirements and the like. In this case verticals can readily be made of 2x6, but will require additional lateral staying. Verticals are usually stayed every four feet in height with 1x4″ ribbons in both directions.

ARTICLE 2.

Leveling Up Centering.

Leveling should be done, using an architect's or an engineer's level.

Evidently the fewer verticals there are the more readily can the form be leveled up and placed in proper condition for casting concrete. After leveling up, the wedges should be nailed so that there will be no slipping. The vertical studs should be stayed along the line of the joist at the top and longitudinally and transversely midway for stories ten to twelve feet in height, so there will be no danger of the stud kicking and the centering going down should a heavy car run off the track. Where the area to be centered is large it sometimes pays to cleat the sheathing in cleats two feet or more wide. This eliminates the necessity for the larger part of the nailing to the joist and enables the taking down of the forms a little more readily.

Wide boards should not be used for sheathing for the reason that they curl and split badly in the sun and swell excessively when wet. For that reason the writer prefers 1x6 square edge fencing. Yellow pine and wood which will stand considerable hard usage is preferable to hemlock or the softer grades of white pine.

ARTICLE 3.

Wetting Down Wood Centering.

Where wood sheathing is used for the forms it should be thoroughly wet down from one to two hours in advance of placing the concrete, to give the timber, which has probably dried out in the sun, a chance to swell and close the cracks so that there will be the least possible loss of cement grout as the casting proceeds.

ARTICLE 4.

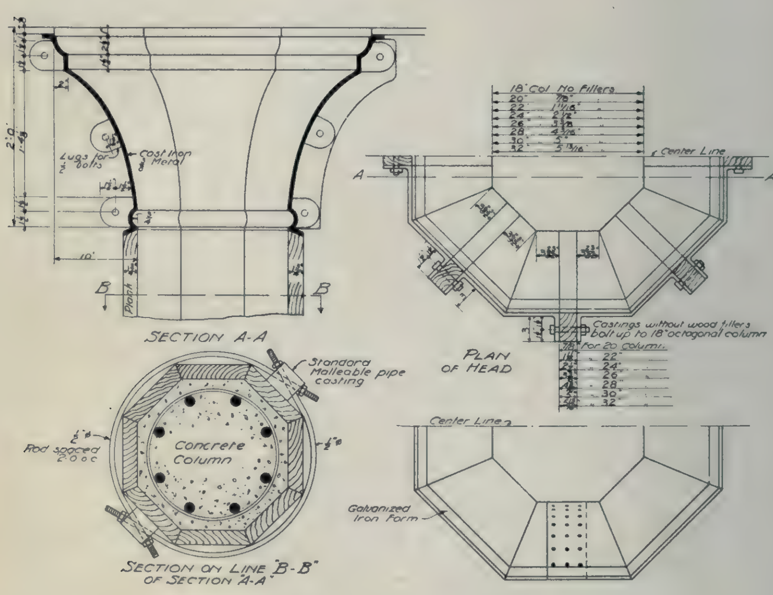
Inspection of Centering Before Casting Concrete.

As a general rule the foreman should be instructed to inspect carefully all centering before starting to pour the concrete for the reason that many of the stays and sometimes some of the verticals are left out temporarily for convenience in erection, with the expectation of putting them in before pouring concrete commences.

ARTICLE 5.

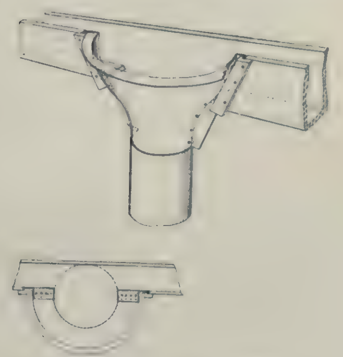
Column Forms.

For octagonal forms we have adopted the standard shown in the figure, with a cast iron or sheet metal form for the head, adjustable. The column box is bound together by ½″ rods bent in semi-circular form, with a long thread and nut at the end. These are passed through standard malleable clamps used for wood stove pipe and screwed up.

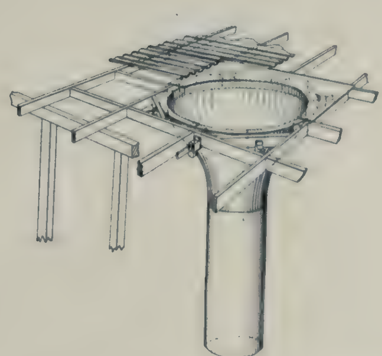


Detail of form for octagonal columns.

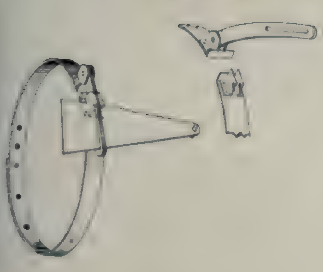
Details of Sheet Metal Forms as manufactured by Lefebvre-Deslauries.



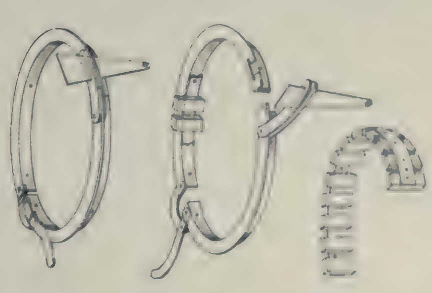
Detail Beam and Capitol.



Detail showing Apron when using Corrugated Sheets for Slab Forms.



Band Details.



Filling Rings.

Another method of making up column forms is to use sheet metal. These are shown in cut below.

This is one of the most economical types of column forms. It is readily handled and weighs but little and costs but little to transport and is reasonable in first cost.

In general, a light sheet metal form consists of sections which are adjustable by being lapped and are held rigidly by heavy bands of quarter-inch metal at intervals of about two feet.

View from below, showing centering.

Lindeke-Warner Co., wholesale dry goods building. Erection carried on at the rate of a story every ten days, including exterior walls, concrete floor slabs, strips and strip filling ready for finished flooring.

ARTICLE 6.

Sheet Metal for Slab Forms.

To save the cost of sheathing and facilitate rapid handling we have used a large amount of corrugated steel in place of fencing for decking.

Ceiling of this type is shown in figure. This type of floor centering is not suitable where it is desired to plaster, but for a wholesale building and in fact any kind where special decorative finish is not desired it is substantial and neat.

Cost of handling sheet metal is about one-third that of laying boarding. Greasing it with paraffine oil prevents the concrete from adhering and facilitates ready removal and re-handling.

Ziegler Building, Milwaukee. Sheet metal forms used for slabs.

ARTICLE 7.

Advantages that are claimed for sheet metal forms are as follows:

That the sheet metal holds the moisture and prevents the concrete from drying out too rapidly. It prevents loss by leak-

age of the liquid cement mortar, as is sometimes the case where board forms are used, and leaves a clean, smooth job.

The sheet metal centering can be used over and over again and should it be battered out of shape it is a comparatively inexpensive matter to re-press the sheets. At first cost it is at a disadvantage as compared with wood centering, but in the long run it is much cheaper for the reason that it is lighter, requires less labor to handle and involves less labor in carting from point to point.

ARTICLE 8.

Improper Specifications for Centering.

Many architects have a totally erroneous idea as to the proper requirements for centering. For example, we frequently see an architect specifying matched and surfaced lumber for forms, with the vague expectation that by so doing he will get an exceptionally smooth job. Unless the lumber is over two inches thick, which would involve an unreasonably great expense, the tongue and groove will be soon broken, ragged joints and edges will be a frequent rather than a rare occurrence, and on the whole the work will not present as smooth an appearance as though ordinary square edge fencing was used for the work.

If it is required that the work be finished with a smooth surface the most inexpensive proposition is to give it a skin coat of plaster as recommended in chapter covering the subject of plastering on reinforced concrete.

ARTICLE 9.

Partial Removal of the Forms.

It is evident that the earlier the centering can be removed and used in the upper stories the less material it will be necessary to use in handling the work and the lower the cost if successfully executed. In the mushroom system it is customary in good dry weather to remove the forms in from twenty-four to forty-eight hours from columns. In this type of construction columns carry little weight until after the removal of the slab forms, and where handling the work in this manner a very much smaller number of forms can be used on a large job. Where, however, beam and slab forms are used the column gen-

erally supports the beam boxes and the writer is not in favor of removing the centering in part, but prefers to see the whole left in except perhaps a few of the stays until the concrete has thoroughly cured.

In the illustrations under the rapidity of erection of reinforced concrete a number of examples are shown which clearly indicate the number of floors under which the centering is left in the conduct of work under favorable conditions.

Handling and making up of forms is more a question of craft than of figures. As to the question of ingenuity the brightest engineer can as a rule learn something from any foreman and even a good carpenter that he comes in contact with in this line of work. Frequently, however, we see workmen who lack ingenuity and a conception of the simple requirements of form work. For example, we occasionally see a gang of carpenters putting up an expensive braced form for a thin wall, where all that is necessary to do is to set up the cleated boards and tie them together with No. 10 wire. The pressure on the two sides balance and the need of bracing is practically nil.

Special forms, such as are used for chimneys, are very advantageously made up with sheet metal and arranged to be slipped upward as the work advances. It is hardly, however, within the scope of this work to go into special constructions of that character.

ARTICLE 10.

Suspended Ceilings.

Frequently a slab is put up where it is desired to suspend a ceiling below, either to conceal pipes, flues and the like, or as insulation for the roof. This is readily arranged in the following manner:

Take ordinary ¼″ round wire, make a 3″ loop on the upper end and drop it through a hole in the form. It will then be anchored in the concrete as soon as the concrete is cast, and the free end may be used to tie up angles, tees or groove irons which may be used for the ceiling frame.

CHAPTER XVIII.

Cost of Work.

ARTICLE 1.

General Statements.

The owner or purchaser desires to know the relative cost of various types of construction for his building. General information is what he wishes and that we will endeavor to take up in article 1 of this chapter. Later the detail or itemized estimate of cost desired by the general contractor will be discussed.

In general, we may say that reinforced concrete is especially adapted to heavy construction; that nothing can compete with it, not even timber, for the heaviest warehouse work where spans are eighteen to twenty feet centers and working loads of 250 to 500 pounds. This general statement holds true throughout the eastern and central portions of the United States and Canada, though it may not be true in some districts in the west where good fir timber costs as little as seven dollars a thousand but even there when the insurance rate is taken into consideration on the building and the goods stored, the concrete structure is the cheaper in the end.

ARTICLE 2.

Adaptability.

Reinforced concrete is not adapted for long spans and light loads. For instance, in a span of fifty or sixty feet in a shop or factory building having only a light roof load, reinforced concrete is not an economical material to use. Structural steel costs far less and is generally employed. For bridges of long spans and light loads reinforced concrete is not economical in first cost. Where the loads are heavy, as for city bridges and the grade such that there is opportunity for ample rise, a reinforced concrete arch may be built at a cost not greatly exceeding that of a good structural steel bridge and when the maintenance charges are considered the concrete will be the least expensive.

For the office building and ordinary business blocks in general, reinforced concrete will save the owner from one-half to three-quarters of the cost of a structural steel skeleton.

Reinforced concrete is particularly well adapted for school buildings. The difference in cost between the ordinary timber floor and a reinforced concrete floor will frequently not exceed ten or fifteen cents per square foot. With this fact in mind it is really astonishing to note how frequently dangerous fire traps are erected to serve as school buildings in which expensive and ornamental exteriors have been used when plainer buildings with fireproof construction could be put up for the same money.

The architect for a school building, in order to make a show, frequently specifies a fancy brick exterior, terra cotta or stone trimmings and other external frills and then economizes in the interior construction of the building by the use of wood joists 1½ or 1⅜″ thick by 16″ deep covered with ⅞ rough floor and ⅞ hardwood finished floor, wood lath and plaster on the under side, electric wires, heating flues, etc., between the joists. This is a construction in which if a fire once started there would be hardly time for the occupants of the building to escape before the collapse of the floor and loss of life incident thereto.

The Collinwood disaster well illustrates this fact. The state of Wisconsin has passed a law making it compulsory to put up school buildings of fireproof materials and other states may well follow her example.

For buildings subject to the vibration of heavy machinery, concrete steel construction has many advantages. Properly designed, the joints (connections of floors to columns) are far more rigid than in any of the old types of construction hence a rigid building costs least in concrete steel.

The Forman-Ford Company, plate glass dealers, etc., make the claim that twenty-five per cent more work is done in their cutting and polishing department in their new concrete building with the same men than in the old timber framed structure previously occupied due to the increase in rigidity of the structure.

ARTICLE 3.

Aggregate.

In many localities crushed stone is expensive and we can find a good gravel bank. Or, in the absence of that, smelter

slag makes a good aggregate. Frequently we find bank gravel which runs somewhat high in sand. Then it is a question as to the uniformity of the material. If the proportion of the sand is constant and too high we can use more cement to keep the proportions of cement to sand down to those required rather than go to the expense of screening the gravel.

ARTICLE 4.

Analysis of Items of Cost.

In arriving at a detail estimate of cost we have the following items to consider:

Basis of Labor.

Materials and cost of handling:

Concrete, Unit price	Cement Sand, Stone (or gravel) Water Common labor Cost of plant	Quantities and base prices.
Steel	Cost of metal Cost of unloading Labor, cost of bending (union or common) Labor, cost of placing (union or common)	
Centering	Cost of lumber Cost of framing beam boxes, columns, etc. Cost for erecting and rehandling Slab forms, beam forms and column forms.	

Season of the year.

Floor finish or strip fill.

Dead expense.

General data on costs per foot of floor and items entering into it.

ARTICLE 5.

Labor, Unit Prices, Quantities of Material.

Under the general heading of basis of labor the contractor must consider, first, the wages per hour; second, the character and efficiency of the labor, whether the labor is union or non-union, probability of strikes and delay of work into the unfavorable season when artificial heat must be used.

Where trade unions are strong the specialist in reinforced concrete can never tell when his work will be tied up by some disagreement between master plumbers and walking delegates or other trades with which he has no relation whatever further than that a sympathetic strike may be called without notice or grievance at any time and his operations brought to a standstill.

This condition means idle equipment and sometimes cost of heating materials and may mean readily an additional cost of five to ten per cent in the work.

Where unions are strong they frequently endeavor to force the employment of totally inexperienced and ignorant men on the constructor. Thus they will wish to force the employment of structural iron workers in place of trained laborers in placing the reinforcement; to prevent the employment of common labor in removing centering and the like.

In Chicago the bricklayers' union demands that the contractor shall keep employed an extra brick foreman who is supposed merely to watch the placing of concrete wherever there may be brick work on the job.

While unionism ought and should prove of benefit to employer and employee alike, when the motto is efficiency and good wages for skilled service; when the union organization is degraded to the point of demanding that unskilled men shall be employed simply because they are members of its organization, it loses both public sympathy and support. In many cases concrete is used for exterior walls where brick, were it not for the short-sighted union leadership, should legitimately be used from the true economic standpoint.

For purpose of discussion and comparison we will take the following costs: Common labor, $2.25 per day of ten hours; carpenters, $3.00 per day of eight hours; steel to be placed by common labor at $2.25 to $3.00 per day.

Unit Price of Concrete.

1-2-4 Mix.

Material for one cubic yard of wet mix concrete:

Cement....................	1½ bbls.
Crushed stone..............	.9 cubic yards.
Sand......................	.45 cubic yards.

Where a crusher run, including dust of good hard crystalline

stone is used the sand may be readily reduced to one-third yard.

The above amounts required to make a cubic yard in place of wet mixed concrete may vary somewhat on the character of the crushed stone or gravel, but for estimating purposes they are conservative.

1-1½-3 Mix.

Material for one cubic yard of concrete, wet mix:

Cement	2 bbls.
Sand	.43 cubic yards.
Stone	.85 cubic yards.

1-3-5 Mix.

Material for one cubic yard of concrete, wet mix:

Cement	4½ sacks=1.125 bbls.
Sand	.52 cubic yards.
Stone	.85 cubic yards.

Labor of Handling.

Given an ordinary equipment such as a half-yard Smith, Cube or Ransome machine the labor cost of handling concrete may be stated as follows.

Wheelbarrow gang, from $1.25 to $1.50 per cubic yard, including cost of coal or gasoline for the engine. In walls, footings or where there is quite a mass this may be reduced to $1.00 per cubic yard.

On a large job where one-half yard cars are used and overhead bins for handling the aggregate by gravity, the labor cost may be reduced to 35c to 40c per cubic yard. To this must be added, however, the cost of fitting up the plant which will increase this figure to sixty or even seventy cents per cubic yard.

The labor costs will increase or decrease as the price of common labor is above or below twenty-two and one-half cents per hour, figured upon.

Cost of cement varies with the market and distance of the work from the nearest mill from eighty cents or a dollar per barrel to two or three dollars.

Cost of crushed stone varies with the locality and distance of the work from railroad or crushing plant.

In Minneapolis and St. Paul, from $1.25 to $1.75 on the

work. Milwaukee, $1.25 to $1.50. Ohio river points, $1.00 to $1.25, washed gravel, etc.

Cost of carting and hauling must be investigated in each individual case. In many of the smaller towns good concrete gravel can be secured as low as thirty to fifty cents per cubic yard and in order to give a clear idea as to the general questions of cost these variables must be carefully considered and investigated by the bidder if figuring reasonably close.

In securing this essential information the conservative business man will secure quotations in writing, especially when not personally acquainted with the reliability of the parties quoting; then if the work is secured he may at his option hold the bidder to his price or seek redress by suit.

In giving the foregoing average values it should be noted that the cost of placing varies eight to ten per cent with the character of the reinforcement. Where there are numerous beam boxes and stirrups and increased work of puddling the concrete, the cost may readily run five or six per cent above the average while where there is plain flat slab such as the mushroom system the cost will readily run five or six per cent lower than the average given.

ARTICLE 6.

Cost of Steel.

Medium Steel, Open Hearth or Bessemer, Manufacturers' Standard specification is at present writing at $1.20 base, Pittsburg.

The base price is given in all the engineering and iron trade papers. All bars from ¾″ rounds to three inches are base. Smaller bars are sold at base plus half card extras.

The following is the standard steel classification:

	Extra	
¾″ to 3″	Base	Rounds or square.
⅝″ to 11-16″	.10	
½″ to 9-16″	.20	
7-16″	.40	
⅜″	.50	
5-16″	.60	
¼″ to 9-32″	.70	

1 to 6″x3/8 to 1″............Base } Flats and heavy
1 to 6″x1/4 and 5-6.......... .20 } bands.

The above are full extras and such sizes as we can ordinarily use to advantage in concrete steel construction.

In figuring, take base price plus half extra plus freight to destination. Freight rates to all points in the United States and the Dominion of Canada are given in compact form in a book published by the American Steel & Wire Company.

Cost of deformed bars rolled to standard specification at the present writing, two to three dollars per ton above plain bars. Special reinforcement sold with design from twenty to fifty dollars per ton additional, whatever the purchaser can be induced to pay.

ARTICLE 7.

Cost of Bending.

Using medium steel with proper equipment rods for the mushroom system can be bent cold for fifty to sixty cents per ton; where high carbon steel is used and rods are heated one and a half to two dollars per ton.

Beam rods, such as are used in Turner beam system, can be bent for from two to two and a half dollars per ton; where more complicated bends are employed from two and a half to three and a half per ton.

Cost of placing steel, including handling and bending, in the writer's work has run from six to ten dollars per ton. With a beam system beams spaced four to six feet centers a cost of ten to twelve dollars would be a fair basis upon which to figure.

Cost of Hooping for Columns.

Spirals made at the shop, six cents a pound in place.

Spirals made at the job, about three and one-half to four cents.

Hoops base price plus freight plus a cent per pound for shop work and half cent for erection. Price on hoops is based on electric welding, allowing for waste.

Estimate on spirals based on $1.20 base for steel, allowing 35c freight in car lots and less than car load rates on fabricated spiral.

In other words, it is much cheaper to bend the spirals on the

job and while the work may not be done with the high degree of uniformity with which it is executed in the shop it is sufficiently accurate for practical purposes if there has been reasonable care exercised in its execution. On the other hand, the made-up spiral is usually more nearly in line than a shop-made which has been shipped and most always kinked in shipping.

ARTICLE 8.

Cost of Centering.

No item in the line of concrete construction is so generally underestimated as the cost of false work for reinforced concrete. In fact, so generally is this the case that the contractor inexperienced in this class of work is more than likely to underbid those possessing both equipment and experience by underestimating this item of cost.

Cost of the centering per foot of floor, including columns and beams, will vary anywhere from six to more than twenty cents per square foot, depending on the following items:

1. The number of beam boxes, whether they frame into each other or into column boxes only.

2. The number of columns for a given floor area.

3. The number of stories or floors that are alike.

The rapidity with which it is desired to push the work and whether the weather conditions are favorable for the prompt removal of the forms.

Where the building has a full concrete skeleton, centering costs generally a third more than where bearing walls are used. Evidently the greater number of stories the more times the lumber may be moved up and used over. Where the work is to be pushed rapidly in cold weather a larger amount of lumber is required.

The practical constructor is inclined to check estimate of cost on the basis of so much per foot of floor for centering for reinforcement and concrete and estimating in this rough way will generally detect an error of more than four or five per cent in an elaborate detailed estimate hence the writer's object in calling attention to the elements noted.

1. Lumber required, nails and fastenings.

2. Carpenter labor of framing beam boxes, column boxes, etc., per thousand feet.

3. Labor of setting plain slab forms.

4. Labor of taking down forms and moving up to upper story per thousand feet B. M.

5. Waste of lumber and value of old centering.

Under one, the amount of lumber required it should be observed that the amount will vary with the type of design. Such a type as the mushroom system, type IV, figure 4, we would have for the sheathing approximately one board foot for each flat foot of floor area. For the joist ¾ of a board foot per flat foot of floor area. For ledgers one-third of a foot, board measure, for each flat foot of floor. For uprights, ⅝ of a board foot for each flat foot of floor. For columns, spacing 18′ centers, from one-third to one-half board foot for each flat foot of floor. Total, about three and one-quarter or three and one-half board feet per foot of floor.

If the work is to be pushed rapidly we would figure, under favorable conditions for centering, not less than two complete floors of centering plus waste. If the weather conditions are unfavorable we should have enough lumber for centering for three to four floors. On a building having eight stories the writer would ordinarily figure enough centering for three floors, plus waste. With the flat slab system there is no waste with the joists as they are simply lapped by and the waste in the boards would amount to about two per cent each time they are used. There will be some waste in the uprights if the stories are of different heights which must be figured in each individual case.

Where a beam system is used the waste will be much greater as the loss from breakage and cutting the lumber to the size of the panels will generally run the waste up to ten to fifteen per cent of the form lumber in each floor, sometimes much more than this. Also the surface contact is increased by the area of the sides of all beams requiring additional lumber.

Cost of Framing.

Labor for framing beam boxes, column boxes, etc., will generally run about twelve dollars per thousand feet B. M. Labor of placing plain slab forms, carpenter's wages, being figured at 37½c per hour, will run about five to six dollars per thousand feet. The cost of taking down the forms and moving them up should run about three dollars per one thousand feet B. M., for the flat slab type and five to seven dollars per thou-

sand where there are a large number of beam boxes, etc. Nails and fastenings are generally a small item.

Where sheet metal is used for the sheathing the cost per foot of laying it and greasing it with paraffine is about one-third the cost of placing boards although the first cost of the metal is considerably higher.

Mr. L. S. Wason, president of the Alberthaw Construction Company, of Boston, at the fifth annual convention of the National Association of Cement Users at Cleveland, Ohio, presented a paper on costs from which the following table is quoted, giving the cost of handling and some very interesting costs of centering. It would be well for the reader to look up this paper which is reprinted in part in the Engineering News, January 14, 1909, and a number of the other engineering papers.

The following table, condensed by the Engineering News, from the original paper, is given as a fair indication of the

TABLE 1.—SHOWING COST OF FORMS AND CONCRETE ON VARIOUS MEMBERS IN REINFORCED-CONCRETE STRUCTURES.

	Forms per sq. ft.				Concrete per cu. ft.						
Location.	Carpenter Labor	Lumber	Nails and Wire	Total	Concrete Labor	General Labor	Cement	Aggregate	Team and Misc.	Plant	Total
PLAIN CONCRETE COLUMNS											
ffice building, Portland, Me.	$.133	$.039	$.001	$.173	$.064	$.004	$.087	$.084	$.012	$.022	$.273
oal pocket, Lawrence, Miss.	.057	.024	.001	.082	.166	.003	.073	.041	.008	.016	.307
Iill, Southbridge, Mass.	.097	.082	.002	.181	.073	.056	.107	.035	.027	.030	.328
Iill, Attleboro, Mass.	.093	.022	.001	.116	.110	.014	.062	.038	.013	.034	.271
Iill, Southbridge, Mass	.080	.056	.001	.137	.108	.048	.100	.037	.013	.034	.340
oal pocket, Hartford Ct.	.098	.047	.002	.147	.089	.043	.069	.055	.017	.013	.286
arage, Brookline, Mass.	.071	.051	.002	.124	.070	.028	.072	.058	.041	.020	.289
Varehouse, Portland, Me.	.118	.016	.001	.135	.087	.027	.087	.070	.039	.025	.335
extile mill, Lawrence, Miss.	.061	.013	.001	.075	.095	.019	.109	.027	.018	.015	.283
ighest	.133	.082	.002	.181	.166	.056	.109	.084	.041	.034	.340
owest	.057	.013	.001	.075	.064	.003	.062	.027	.008	.013	.271
verage of 9	.082	.036	.001	.130	.096	.027	.085	.049	.021	.023	.301
REINFORCED-CONCRETE BEAM FLOORS.											
ighest	.165	.107	.004	.275	.186	.035	.194	.101	.052	.055	.470
owest	.037	.027	.001	.067	.047	.004	.071	.037	.007	.010	.202
verage of 18	.070	.045	.002	.116	.111	.020	.106	.063	.025	.024	.354
FLAT SLAB FLOORS.											
ighest	.078	.039	.003	.118	.146	.017	.109	.084	.026	.039	374
owest	.067	.037	.001	.106	.043	.004	.087	.053	.012	.010	.252
verage of 3	.071	.038	.002	.111	.097	.009	.096	.070	.019	.024	.315
REINFORCED-CONCRETE SLABS BETWEEN STEEL BEAMS.											
ighest	.110	.071	.003	.184	.144	.048	208	.080	.064	.046	.428
owest	.028	.012	.001	.049	.073	.005	.076	.026	.004	.010	.272
verage of 13	.061	.032	.002	.095	.102	.019	.128	.068	.024	.017	.359
BUILDING WALLS ABOVE GRADE											
ighest	.136	.073	.005	.176	.146	.052	.105	.187	.077	.055	.446
owest	.046	.016	.001	.079	.042	.004	.034	.043	.007	005	174
verage of 17	.085	.036	.002	.128	.090	.016	.073	.076	.025	019	301
FOUNDATION WALLS											
ighest	.134	.048	.004	.193	.213	.037	.203	.116	.057	.040	.599
owest	.032	.009	.001	.056	.040	.002	.038	.027	003	010	148
verage of 14	.068	.033	.002	.103	.076	.015	.080	062	.019	.017	.269
FOOTINGS AND MASS FOUNDATIONS.											
ighest	.119	.077	.003	.198	.081	.020	.098	.099	.013	.049	.275
owest	.016	.006	.001	.018	.025	.001	.047	043	.003	010	.181
verage of 10	.057	.034	.002	.093	.045	.007	.071	.077	.007	.021	.229

variation in cost of different designs and different conditions. The author states that only typical types are given where the items of cost were accurately known. Enough are given for a fair average except in the case of long span flat slab which appears to him by comparison a recent type of construction.

By reference to the general averages on form work in the foregoing tables the cost of forms per square foot of surface contact, namely: Columns, $0.13; floors with reinforced concrete beams, $0.116; flat floors without beams, $0.111; short span slabs between steel beams including the fireproofing on the side of the beams, $0.05; walls exposed to view above ground, $0.093; the writer believes are all higher in price than usually believed to be a fair cost by the majority of builders. It is upon the success of handling forms that good results financially depend. In regard to concrete, labor is the variable item which must be carefully considered. Any one of intelligence can make a careful estimate of the materials to be used but note the average prices of labor per cubic foot of concrete, namely: For columns, $0.123; beam floors, $0.131; flat floors, $0.106, floors between steel beams, $0.121; walls, $0.106; foundations, $0.091; and mass work in connection with buildings, $0.052; not until the last item is reached is a price obtained in experience which, according to the observation of the writer, the majority expect to obtain in building work in general. Many men who have had wide experience in handling large quantities of concrete in mass have at times attempted a lighter type of construction and have been greatly surprised at the large expense connected therewith. It has come to the writer's notice a number of times that men with this experience have added fifty to one hundred per cent to the cost of mass work and felt that they were amply covered for light structural work.

Table II is an exact copy of a "master card" which gives the complete financial history of the job, when it is finally completed. The first column, which is blank, is occasionally used for an estimate of the first cost, the proposal including the profit as well as the estimated actual cost. It will be seen that on some items, a loss was incurred, as well as a profit on others, showing that it is difficult, even on work which a company is fairly experienced on, to reach the right price on everything, and also that when slight changes are made by the owner or architect they often entail heavy loss even though the changes

appear to be extremely trivial. Take the case of the external walls. The owners furnished the window frames and sash, which were all of metal. The original design was for a frame with two sash, which could easily be put into a six-inch wall.

TABLE II.—TYPICAL "MASTER CARD."

Job No. 747. Date, May 24th, 1906. Mill, Tappan Bros., Attleboro, Mass

	Proposal.	Actual Cost	Per cu. ft.	Profit	Loss	%
Total	$35,164.55	$31,330.48	....	$3,834.07	....	11
Excavate	790.00	823.18	$0.021	......	$33.18	..
Footings and Fn	1,738.00	1,033.57	.137	704.43	....	..
			Per sq. ft.			
Exterior walls	1,955.00	2,162.02	.190	......	207.02	..
Wall and Fn. centers	1,520.00	3,630.08	.125	......	2,110.08	..
Floors $6\frac{3}{4}$ ins. thick	8,883.00	6,544.16	.339	2,338.84	....	..
Roof $5\frac{1}{4}$ ins. thick	2,869.00	1,713.51	.237	1,155.49	....	..
			Per lin. ft.			
Columns, 20 ins. × 20 ins	832.00	676.65	1.470	155.35	....	..
Stairs	883.00	910.35	.912	......	27.35	..
			Per sq. ft.			
Tool surface	469.00	636.53	.056	......	167.53	..
Ornaments and cornice	348.00	164.33	....	183.67	....	..
Ventilators on roof	44.00	35.64	....	8.36	....	..
			Each			
Set windows and door frames	852.00	729.99	2.19	122.01	....	..
			Per sq. ft.			
Interior partitions	1,770.25	1,656.35	.189	113.90	....	..
Bolts and iron work	253.00	257.06	....	......	4.06	..
Stair railing and grill	387.00	654.00	....	......	267.00	..
			Per M.			
Screeds and settings	1,086.00	835.12	52.17	250.88	....	..
2-in Spr. plank and laying	2,839.00	1,431.69	33.30	1,407.31	....	..
$\frac{7}{8}$-in. maple	1,738.00	1,788.88	89.44	......	50.88	..
Motor shaft	379.50	533.19	98.89	......	153.69	..
Motor shaft found	98.00	70.07	....	27.93	....	..
Roofing and conductors	1,255.00	1,026.06	....	228.94	....	..
			Per sq. ft.			
Paving	1,009.00	647.54	.094	361.46	....	..
Retaining wall centers, per sq. ft.	....	....	.211	......	....	..
Retaining wall, concrete per cu. ft.	429.00	316.90	.175	112.10	....	..
Painting	400.00	375.00	....	25.00	....	..
Steel footings and walls	300.00	218.91	....	81.09	....	..
Plant, frt., etc	1,860.00	2,271.73	....	......	411.73	..
Bond	100.00	120.00	....	......	20.00	..
Extras	77.80	67.97	....	9.83	....	..

They later decided, for greater fire protection, to use four sash. This required an eight-inch wall instead of a six inch, and the form work on the inside had to be built inward and then the space under the windows paneled to save material. To save making a very narrow panel at the side of the window, which would cost more than the concrete saved, the space was filled up solid so that the columns appear to be wider than they were actually figured. This slight change, which did not appear great at the time, when the job was entirely complete showed that the concrete on the walls showed an actual loss instead of profit and that the form work cost more than twice what was originally estimated that it should cost.

ARTICLE 9.

Season of Year.

The season of the year has to be considered in its relation to the cost of reinforced concrete work. In the summer season when the concrete dries out rapidly the forms may be removed every ten to twelve days, while in the fall and early spring during frosty weather the water must be heated or the forms left in place longer, requiring more lumber for centering. In the winter when the materials must be heated by artificial heat and artificial heat used in sweating out the concrete, the cost of work will be increased from ten to twelve per cent. Additional cost of merely heating the water is of course small. In the chilly weather of fall or spring good results may be frequently obtained merely by turning the exhaust steam into the water barrel and warming the water up so the concrete will set quickly notwithstanding the chilly temperature.

ARTICLE 10.

Floor Finish or Strip Fill.

Cost of placing strips and filling between them will generally run from three to three and one-half cents per foot of floor while a good cement finish, 1¼″ thick can generally be laid for forty to fifty cents per square yard.

ARTICLE 11.

Dead Charges.

No contracting firm can do business without a considerable general expense, which must be distributed over all work executed by them. This expense includes office expense, advertising, soliciting work, estimates on not only the work taken but the work which the concern fails to secure, depreciation of the plant, freight, storage and equipment, the cost of keeping the organization together in slack periods. This expense may readily vary with various concerns from five to seven per cent of the cost of the work executed. In addition to this dead expense and the actual cost of labor there must be included the item for liability insurance which the contractor cannot afford to neglect to carry. Frequently the owner requires a surety

bond for the faithful execution of the work on the payment of bills, the cost of which must be added to the incidental charges in the estimate of cost.

ARTICLE 12.

General Data on Cost.

The architect is in the habit of figuring the building as so much per cubic foot. For heavy warehouses with the plainest kind of finish and large size the cost per cubic foot may run as low as six and one-half to seven cents up to ten and twelve cents for the smaller size buildings with office fixtures, plumbing and the like. No approximate cost per cubic foot of value can be given for office buildings, hotels and the like, since this item would vary greatly with the character and difference in the quality of the finish, fittings and the like.

For the concrete end of the building, however, a rough approximate estimate can be made very readily by figuring a unit price per square foot of floor area. In a large building of six or seven stories having a floor area of twenty to thirty thousand feet, panels approximately eighteen feet square, labor as outlined, sand at $1.00 per yard, cement $1.20, crushed stone $1.40, capacity of floors three hundred pounds per foot; rough slabs, columns and footings may be erected at an approximate cost to the contractor of about forty cents per square foot of floor area. Where the building is narrow and there are more columns in proportion to the floor area on the same basis fifty cents per square foot would be a reasonable price.

Reduction in the floor load carried makes a relatively small reduction in the cost of the construction, since the centering would be the same for the light and the heavy building.

Where the load is increased fifty per cent above these requirements the additional cost would be increased over eight per cent. While doubling the load would not increase the cost over about ten or eleven per cent.

This is the general type of information the shrewd contractor carefully figures out for himself and which enables him quickly and accurately to check up estimates made by his assistants or even to take work on an approximate estimate of this kind without going into details. The writer himself at one time when busy took a $60,000 contract on a twenty-minute

estimate based on a computation only of the floor area and a knowledge of the conditions covering labor and cost of materials.

Where there are plain reinforced floors resting on walls and the panels are of large size such as in court house work and many other public buildings and where gravel can be cheaply obtained the cost per foot of floor may run as low as twenty-five cents per square foot, the writer having taken contracts for the fireproofing in court houses on the basis of less than thirty cents per foot of floor area and made money subletting the work, furnishing designs and reinforcement. In other localities forty cents per foot under less favorable conditions would be a reasonable figure.

CHAPTER XIX.

Permanence of Concrete Construction.

ARTICLE 1.

Concrete Made with Proper Materials.

The best grade of Portland concrete made with the first class cement selected aggregate, properly mixed and cured is indeed a most permanent material, fully justifying all that can be said in its favor. It will withstand the action of the elements equal to stone such as granite and quartzite, will withstand the heat of fire better than granite and while in small samples is not equal to the granite in point of strength, in large masses it may be said that it may be depended upon with great certainty due to the fact that there are no seams, flaws or planes of weakness which we are liable to find in masses of natural stone.

A good concrete increases in strength with age and grows harder and stronger as time continues. The increase in strength being rapid for the first three months and continues at a gradual decreasing rate for the next six or eight months, and then very slowly as time goes on, perhaps through a period of twenty-five years or more.

Where the concrete, however, is not made from suitable aggregates, is not properly mixed and cured, it is by no means a permanent material when exposed to the action of the elements. Concrete having for an aggregate a soft stone, such as some of the oolitic lime stones, shale or one which is made with sand which is fine and containing considerable clay will inevitably be affected materially by frost in the severe climate of the north.

ARTICLE 2.

Concrete Made with Improper Materials.

In building work the concrete is kept under cover and in the main protected from the elements, and hence some contractors have an idea that this being the case almost anything at all can be utilized as aggregate. Thus cinders in which there is quite

a large percentage of ash, partly burned coal and the like, have been used in some cases and with exceedingly bad results.

The writer had one case come to his attention in which the concrete had been made from cinders from Southern Iowa coal. The concrete after it was cast in the form of slabs expanded to such an extent that it pushed the face brick out of the side of the building and the slabs checked and cracked to a considerable extent due to this same action.

The writer has just noted in the Engineering Record* the following article by Mr. D. B. Butler:

"EXPANSION OF CONCRETE MADE WITH COKE BREEZE."

"On account of a number of failures of roof and floor slabs, made of coke breeze concrete, which were called to his attention, Mr. D. B. Butler undertook a series of experiments to determine the expansion of such concrete since an examination of the faulty structures indicated that such action was responsible for the failures. His conclusions were presented in a paper before a recent meeting of the Society of Architects, England, from which these notes are taken.

"In nearly all samples of so-called breeze concrete examined by Mr. Butler, a very considerable quantity of material other than pure coke was noticeable in the aggregate, such as clinkers, stones, shale and ashes, together with, in some instances, a noticeable amount of coal. Whatever may be the disadvantages of other extraneous material found in breeze, coal is not, in Mr. Butler's opinion, a desirable constituent for concrete; in the first place, on account of its smooth, shiny surface, the adherence of the cement would be extremely poor; in the second place, it is worse than useless as a fire-proof material, on account of its tendency to decompose on heating. The question arose, however, whether apart from being undesirable for the reasons aforesaid, either coke breeze or coal was in any way dangerous as being likely to cause expansion of the concrete.

"The first experiment was of a somewhat rough and ready nature, and was made with coal. An ordinary bituminous house coal was crushed and sifted to about the fineness of standard sand; with this coal a 3 to 1 mortar was made, and two small 2-ounce glass bottles filled with the mixture; one bottle was filled quite full, and the other was filled to within a quarter inch of the top and sealed down with a paste of neat cement, the object of the sealing being to ascertain whether the imprisonment of any hydrocarbons set free from the coal would have any bursting effect. For comparative purposes similar bottles were also filled with a paste of neat cement and 3 to 1 mortar of standard sand.

"The whole of the bottles eventually cracked, with the exception of one filled with standard sand-cement mortar. But while those entirely filled with the coal mortar generally cracked within two or three days, and with a very few exceptions continued to expand until the bottles burst away into several pieces, those filled with the neat cement and the sand mortar frequently did not develop any cracks whatever till several months, and it was the exception rather than the rule for them to continue expanding sufficiently to burst the bottle. Both the neat cement and sand-cement mortar bottles remained perfectly sound after eleven months and then only developed very minute cracks, whereas the coal-mortar

*June 19, 1909, page 767.

bottle was cracked in twelve days and burst right off in forty-two days. This suggests that the cause of the cracking after such protracted periods might be due to unequal expansion of the glass and the mortars at varying temperatures.

"The subsequent experiments were made with rectangular bars 100 mm long and 22 mm square in cross-section, the expansion and contraction of which were accurately measured in the Bauschinger micrometer calliper apparatus. By the use of this a minute variation of 0.005mm, or 0 005 per cent in the length of the prism, may be detected.

"Eight bars of prisms were made with satisfactory cement, four being made with neat cement and four with 3 to 1 standard sand-cement mortar. Two of each series were kept entirely in air and two placed in water after twenty-four hours and kept therein during three months. The test pieces numbered 300 and involved 5,000 measurements.

A noticeable feature of the experiments is that many of the specimens which show very marked expansion when placed under water as soon as set expand very much less when left entirely in air. It therefore seemed a point worth determining as to whether exposure to damp or moisture would in any way affect these air-set specimens at the end of the three months' test, after they had become thoroughly seasoned. One of the duplicate air bars from each series was therefore placed under water, the time elapsing between the date of moulding and placing under water ranging from 91 to 292 days. Immersion had practically no effect upon those specimens which had previously shown no expansion when kept under water, but it caused almost immediate expansion of a very serious nature with those fractions of breeze which had previously developed expansion when placed under water in the first instance. This clearly shows that the expansive agent, whatever it may be, is more or less dormant in the dry air-set block, and only requires to become damped to constitute a serious element of danger.

"Taken as a whole, the experiments as far as they go seem to point to the fact that as regards subsequent expansion there is not much danger to be apprehended from good, clean coke or clinkers, or even anthracite coal, but that some kinds of ashes and furnace refuse are highly dangerous, while any considerable quantity of bituminous coal is absolutely fatal. One noticeable feature of the experiments, however, was, that most of the coke-breeze mortars had a tendency more or less seriously to attack the iron moulds, causing them to rust during the short space of twenty-four hours between the moulding of the specimens and their removal from the moulds. Mr. Butler is unaware if such results have been found to any appreciable extent in actual practice, but samples of breeze concrete sent him for examination a short time ago showed distinct marks of considerable rusting having taken place where the concrete had been in contact with the rolled joists."

Mr. Butler's experiments quoted above coincide with the writer's personal observations. In general, cinder, if fit to use, should be free from ash and should be well burned stoker clinker. Concrete made from good hard clinker has proved a good and substantial fire proofing material.

There is this difficulty in its use, however, that the contractor too frequently furnishes cinders rather than hard clinker.

ARTICLE 3.

Concrete Mixed Dry and Tamped.

Concrete mixed dry and tamped in the old fashioned way is more or less porous, and liable to disintegrate under severe conditions of exposure, as follows: Such that the concrete is soaked with water, frozen and thawed repeatedly. Such conditions may occur in an aggravated form in retaining walls.

The government sea wall at the ship canal, Duluth, made in the old fashioned manner, mixing the concrete dry and tamped is showing the effect of exposure to a far greater degree than we would expect had the work been executed in accordance with the present standard practice.

In this sea wall it should be noted that in the cold season the wall was alternately wet and dry as the waves washed against it; that moisture is alternately frozen and thawed in the exposed surface, and due to the fact that the method of mixing leaves the concrete slightly porous some distintegration naturally results.

In general, the best concrete to withstand such severe conditions is that which is most dense, is strongest, and made from the hardest and most durable stone as an aggregate and with clean, coarse sand.

Where brick or building stone is made of a fairly dry or moist mixture and is not exposed to the severe conditions above outlined it proves very durable material.

The writer's examination of the work of the Canadian Art Stone Company, of Toronto, convinced him that their material made in this manner of selected aggregate weathered unusually well. Only in rare cases there seem to be instances where the facing has broken away from the rough backing used, and this may be charged to careless workmanship, which may occur in all lines.

ARTICLE 4.

Hair Cracks, Map Checks and Crazing.

In troweling a finished surface on concrete the moisture is brought to the surface by the working of the material, resulting in somewhat unequal conditions of moisture and the exposure promoting the rapid drying out of the surface causing what is known as hair cracks, map checks and the like.

These are generally only of very slight depth and mean little in the permanence of the material, providing the concrete is made using good cement and a first class aggregate.

A peculiar fact concerning this defect in concrete finished surfaces is that on some blocks it will not appear at all, while others made under almost identical conditions will be badly affected. Perhaps the difference in part may be accounted for by the thoroughness with which the concrete has been mixed, the time expended in mixing, as well as the conditions of drying and curing.

Concrete which has been thoroughly mixed in a machine for double or triple the ordinary time will be a little stronger than concrete which has only been mixed for fifteen or twenty revolutions. If the mixing is continued for twenty minutes there will be less tendency towards rapid setting and shrinkage and the development of checks and cracks much on the order of the results obtained by skilled mechanics by retempering cement mortar in patching old work.

In the treatment of concrete which is finished with a troweled surface to prevent checking it is desirable, where it is exposed, to keep it protected by burlap soaked in water and to keep the direct rays of the sun from it by an additional cover of canvas. In this way steps and similar work may be executed with the minimum difficulty from this cause, provided care has been used in the selection of both sand and stone used as aggregates.

In the manufacture of cast stone this difficulty is one with which the worker in this field is forced to contend.

In general, cast stone made by the sand mould process will keep its general color better than such natural stone as Bedford, although it may discolor in streaks and blotches known as crazing.

The same remark made regarding the mixing of concrete where the surface is troweled may be applied to this class of work. Efforts made to overcome crazing may be summarized as follows:

First, by the addition of other ingredients to the cement in mixing with the intent to render the material more perfectly waterproof and more uniform in setting.

Second, to coat or waterproof the material after it has been cast, with a compound repellent to moisture.

Third, to remove a thin layer of the surface of the stone and concrete and get below the depth of the hair lines or depressions which form in casting and cause this peculiar marking or discoloration when exposed to the weather.

The first two methods have apparently been successful in somewhat mitigating this difficulty, while the third method has been successful as practiced by the Roman Stone Company, of Toronto. Their method is to use a Carborundum wheel, dressing and tooling the surface therewith.

ARTICLE 5.

Temperature Effects.

Changes of temperature in concrete cause changes in the volume as with all material with which we have to deal. The difficulty which is encountered is the cracking of the concrete as it is brought in tension by change in volume. Massive walls, unless cut at intervals of thirty feet or thereabouts will crack through from this cause. Where openings, such as windows, are cut through a solid wall of concrete cracks are liable to develop at the corners unless the concrete is well reinforced by steel rods crossing the corners in such a manner as to take care of this tension and prevent the development of cracks.

In slabs reinforced in one direction there should be used not less than eight-hundredths per cent of metal for temperature stress if it is expected to prevent the development of unsightly checks.

ARTICLE 6.

Disintegration of Concrete by Oil, Grease, Etc.

In use in factory buildings, machine shops, etc., oil and grease are liable to come in contact with the concrete and it is important to know what effect it will have upon the material. Certain kinds of oils are known to be positively injurious to concrete in the earlier stages of hardening and to disintegrate it to a considerable extent. Where, however, the concrete has had ample time to harden there seems to be little if any damage resulting from lubricating oils such as are ordinarily employed in a factory or machine shop. Where it is desired to use a floor which has not had at least two months in which to thoroughly harden

the writer would recommend coating the concrete with some good waterproofing compound or floor paint, thereby protecting it until after it has had opportunity to become thoroughly cured and hardened throughout.

The question of disintegration of Portland cement briquettes and experiments to prevent it have been quite fully discussed by Mr. James D. Hain, Assoc. M. A. S. C. E., in the Engineering News, March 16, 1905. His conclusions may be summarized as follows:

1. Most oils penetrate concrete mortar, which makes them dangerous.

2. Concrete is more liable to be disintegrated when saturated with oils and fats if not thoroughly set.

3. A good quality of concrete is less susceptible to oil than a poorer quality, such as a porous, poorly mixed or improperly seasoned concrete.

4. Ordinary concrete work is rarely subjected to continued doses of oil. It is more often only occasionally spattered. Disintegration under the latter conditions seems remote, especially in the case of the first class, well seasoned concrete. The concrete floor mentioned is an excellent example; the oil spattered on it perhaps was oxidized or absorbed by the dust, and instead of penetrating helped to protect it.

Last, even though subjected to the equivalent of continued saturation, the disintegration would be long drawn out if the concrete were properly made and well set. Even under ordinary conditions it seems desirable to use a wash for oil spattered concrete to prevent the oil from penetrating it.

Mr. Hain in his experiments tried the following wash: Five per cent solution of alum and a seven per cent solution of castile soap, and also experimented with paraffine. None of these proved satisfactory where the briquettes were immersed in oil.

The following table shows the result of some of these experiments of Mr. Hain:

No. Briquettes made.	Class of Portland Cement.	Mixture Portland Cement and Sand.	Extract Tar Oil.	Whale Oil.	Castor Oil.	Linseed Oil.	Petroleum Oil. (Crude)	Signal Oil.
			Time applied before disintegration.					
18	Stone&clay	Neat......	3 mos....	*	*	*	*	*
12	Stone&clay	1:3 sand...	*	*	*	*	*	*
18	Marl & clay	Neat......	$2\frac{3}{4}$ mos...	*	*	*	*	*
12	Marl & clay	1:3 sand.	*	*	*	*	*	*
18	Slag& stone	Neat......	1 mo.....	3 mos..	4 mos..	*	*	$1\frac{1}{4}$ mos.
12	Slag& stone	1:3 sand..	7 mos....	$4\frac{1}{2}$ mos.	$6\frac{1}{2}$ mos.	*	*	4 mos.

*Sound after applying oil nine months at which tests were discontinued.
All briquettes set seven days in air before applying oil.

Mr. Reid in his work on concrete states that one of the briquettes tested with signal oil was sent to the laboratory of Toch Brothers, Long Island City, and a careful analysis was made. Mr. Maximilian Toch states that a determination of the soluble substances in the briquette showed that the disintegration was due to the formation of oleate and stearate of calcium. To reduce this to its simplest expression, the animal oils contain acids which combine with the lime and crystals of stearate and oleate of lime are formed. It is very likely that these crystals in the process of formation have increased the bulk in the briquette and the bond which has been formed by the lime in the set cement has been totally disintegrated and ruptured. These crystals were isolated and verified under the microscope.

Mr. Toch also states that machine oils are almost all paraffine oils, do not contain animal fats, and hence do not affect concrete.

Silicate of magnesia, sold under the name of fluate, has often been used as a wash to protect concrete against the action of oil. When this wash is applied to concrete, silica is liberated and fills up the pores. The magnesium fluate acts as a binder, and the cement becomes excessively hard after a few months. Limestone and building stone have been treated with this material in Europe with great success. This compound is, however, expensive.

CHAPTER XX

Requirements of Different Classes of Buildings.

ARTICLE 1.

Classification.

The object of this chapter is to touch on the requirements of some special classes of building, those which are common or general types.

First of these, we will take up factories and warehouses; second, mercantile buildings; third, banks and jails; fourth, hotels, apartment houses and residences; fifth, office buildings, and endeavor to deal with those questions which are usually brought up concerning each of these general classes when constructed in reinforced concrete for the skeleton and floors.

ARTICLE 2.

Factories.

The manufacturer is interested in reinforced concrete due to the advantages obtained by its use in the erection of factories, warehouses, etc., which we may state as follows: Good lighting, rigidity under machinery, economy and rapidity of construction, fire-resisting qualities, permanence and freedom from repairs, deadening of sound, uniformity of temperature and waterproof quality of floors.

In the majority of the manufacturing plants the first requisite is that of good light. The difference in this respect between the beam and flat slab system of construction is well illustrated in figure A and B.

Figure B shows the tendency of the ribs to cut off and prevent the distribution of the light through the room. The accompanying figure C shows the State Twine Plant at Stillwater, Minn., which illustrates this advantage to a high degree. Figure D shows floor of the Dill Collins building, Philadelphia.

With any type of construction the lighting may be materially improved by coating the concrete with a white coat of cold water paint. This may be best made with a wash of slaked white lime in a weak size, using sufficient salt to properly preserve the glue.

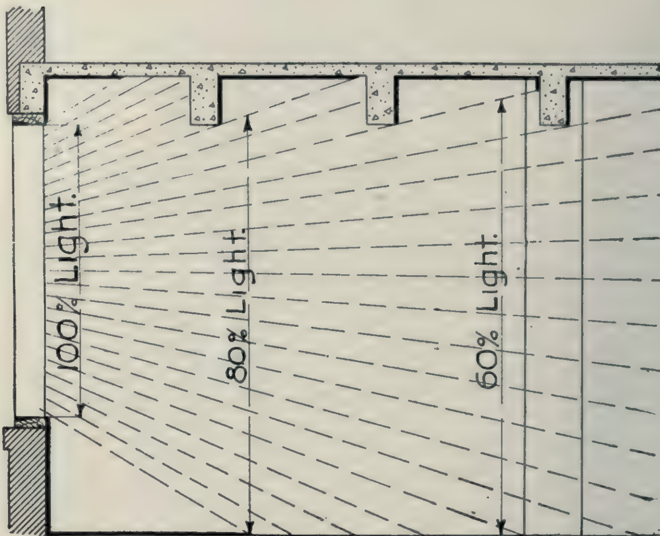


Fig. A. Reinforced Concrete Beam and Girder System. Beams and Girders Block Light Rays.

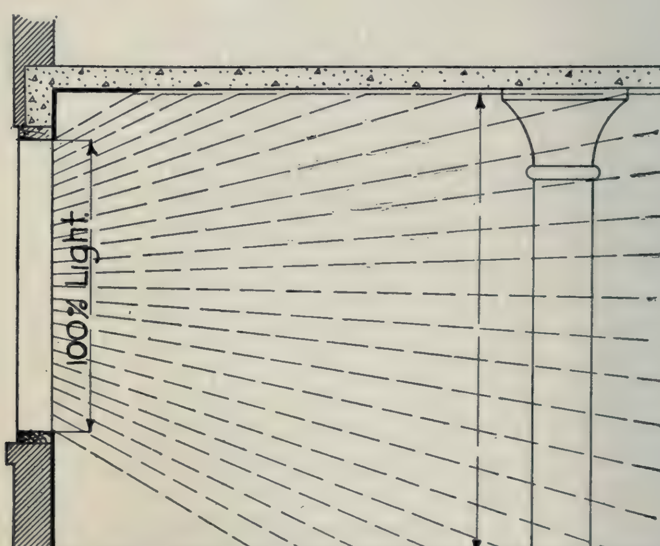
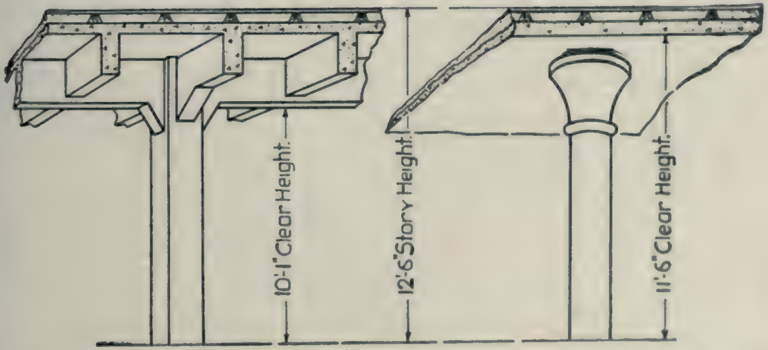


Fig. B. Mushroom System.

Flat ceiling, nothing to block light rays, the light travels along ceiling, lighting interior to greatest distance.

Interior Lindeke-Warner Building.

"Mushroom System," distance from camera to window, 160 feet. Note the flat ceiling, no exposed beams and girders; see the wonderful distribution of light.



Showing 17-inch more head-room in same story heights by using Mushroom System in preference to beam and girder system. Note with the flat slab type there are no ribs to interfere with placing of shafting or sprinkler system.

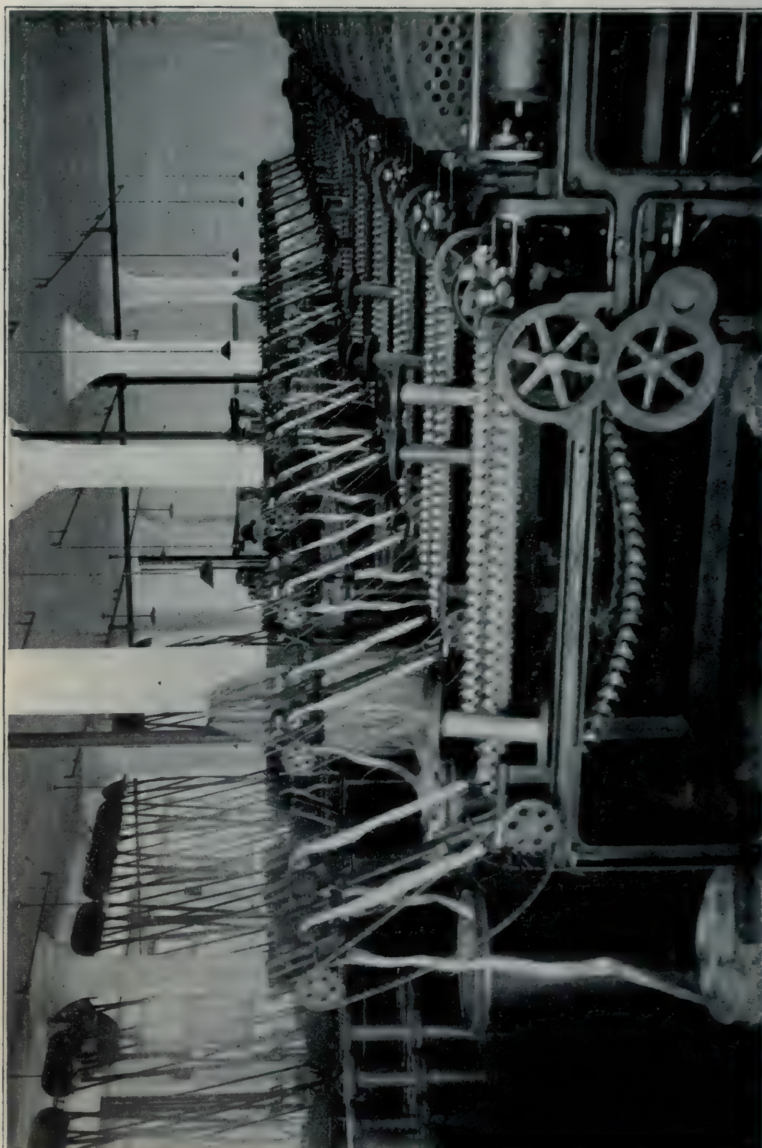


Fig. C. State Prison twine factory, Stillwater, Minn. Note remarkable distribution of light.
W. & J. A. Elliot, Contractors. C. H. Johnston, Architect. C. A. P. Turner, Engineer.

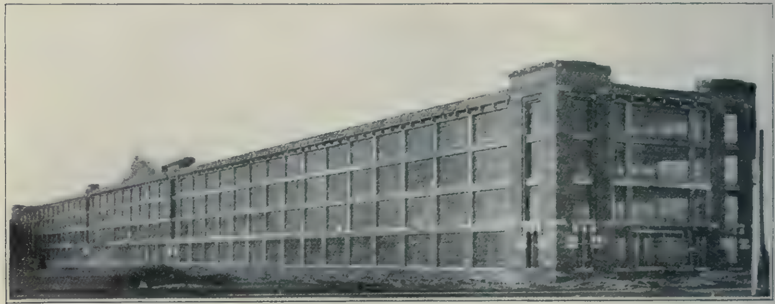
Fig. D. Printing Building, Mushroom System, Philadelphia, Pa.

This coat will not rub or chalk and is one of the best and cheapest cold water paints that can be used. It can be applied with pneumatic spraying machine at a cost of about three and one-half to four cents per square yard of surface covered.

A concrete structure has this advantage in securing good light in the work rooms, that it can be put up in a skeleton frame and the area between consecutive columns filled with glass.

To make the structure more completely fireproof and incombustible, metal frames are, of course, to be preferred to wood sash. These are frequently made of sheet metal, but some of the later types of steel sash may be considered preferable in point of rigidity and ability to be placed in large units at a moderate cost.

We show in the accompanying figure detail of steel sash manufactured by the Detroit Steel Products Company. This



Ford Motor Co. Factory, Detroit, Mich. An excellent exterior, well lighted.

was used in the Ford Motor Company's factory in Detroit, which is an excellent example of general design of exterior walls for this class of building.

Second, as to rigidity: The rigidity under vibration will depend quite largely on the thickness of the material directly under the machines. Whether the floor is made up of thin short span slabs or thicker long span slabs. The long span slab has the advantage that under any impulse or jar due to machinery its vibration is opposed by the reinforcement through a larger area in the slab and a greater mass of material, hence the long span slab, whether between beams or covering area from column to column, is much more rigid under vibratory loads than the thinner short span slabs.

This advantage in point of rigidity is greatly appreciated by plate glass firms, one for whom the writer designed a large structure, making the claim that the men in the concrete building do at least twenty-five per cent more work than their output in the old timber building previously occupied, due to the increase in rigidity of the structure.

Under heavy printing presses there is almost no perceptible

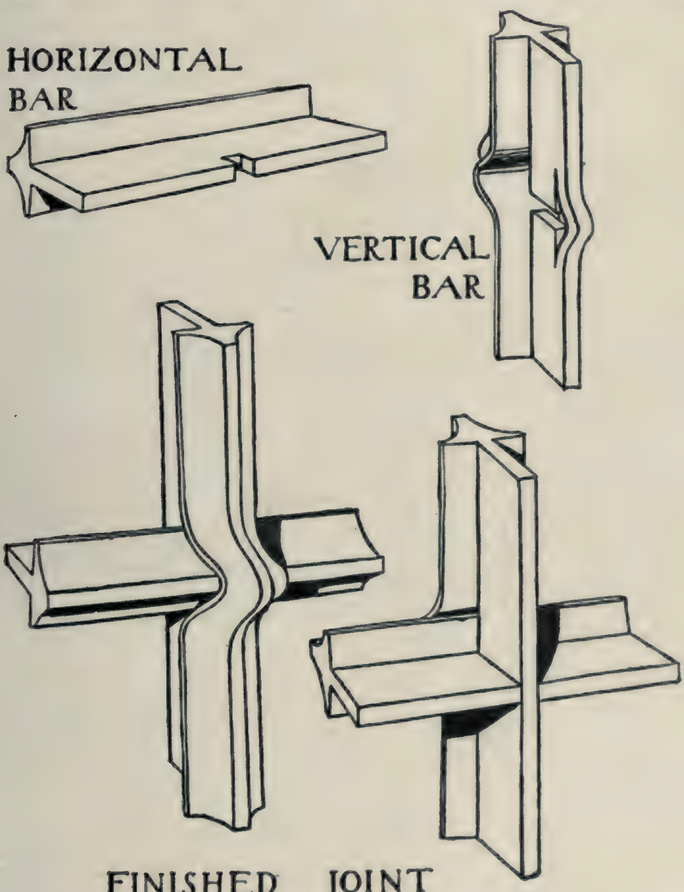


Fig. E. Details of Steel Sash.

vibration in a well designed concrete building, while in an average structural steel and tile construction used in a number of buildings that the writer has investigated considerable sway is caused by the running of the heavy presses.

We have treated the rapidity of erection in a separate chapter and in this connection need only state that this type of construction ranks first, since the materials can be more readily obtained than those for any other type of construction.

As regards economy, wherever the loads are at all heavy, even timber cannot compete with concrete.

Fire-resisting qualities of concrete have been treated under a separate general heading and nothing further need be noted in this chapter except to call attention to the advantage to the manufacturer in the reduced rate of insurance on the building and contents and the almost absolute certainty that no fire can occur which will put him out of business by the more or less complete destruction of his plant.

Loss in this manner may frequently be far more serious to an established industry than the value of the actual material destroyed.

As regards freedom from repairs, compare, for instance, the ordinary cotton mill, in which it is necessary to continually mop the floors to keep the air at the proper humidity to prevent snapping of the threads in working the material, with the resulting decay and rotting of the heavy plank floors, with the permanence that is secured by using reinforced concrete.

Evidently where floors will rot away in the timber construction in a few years and have to be replaced piecemeal, the advantage of concrete can better be appreciated. Here again, the waterproof characteristic of the floors is of value.

Another advantage which is noticeable in going into a machine shop or factory building of reinforced concrete is the fact that the sound is deadened to a great extent. Deafening effect is more perfect the thicker the slab and follows the same laws as regards rigidity of concrete construction. We may further note that concrete is a material which will tend to preserve a more uniform temperature in a building, due to its non-conducting properties. The poor conducting qualities of concrete keep rooms formed by it at a very even temperature. This is particularly noticeable where they are next to the roof and where it frequently happens they are otherwise unbearably hot in the summer and excessively cold in the winter.

For warehouse buildings good lighting is an advantage of considerable importance and the additional clear head room ob-

tained with a given clear story height with a flat slab construction over that with a beam construction gives an advantage from the standpoint of economy of nearly ten per cent of the material in the exterior walls, while the low coefficient of bending in such a flat slab type as the mushroom system gives this type a decided advantage wherever the loads are at all heavy.

ARTICLE 3.

Mercantile Buildings.

In these the question of lighting is equally as important as in factories. The advantage of the flat slab type is the ability to place partitions anywhere that may be desired to divide the space which it may be desired by the owner to rent without regard to position of beams gives this class of construction a considerable advantage over the older beam and slab type.

ARTICLE 4.

Cold Storage Buildings.

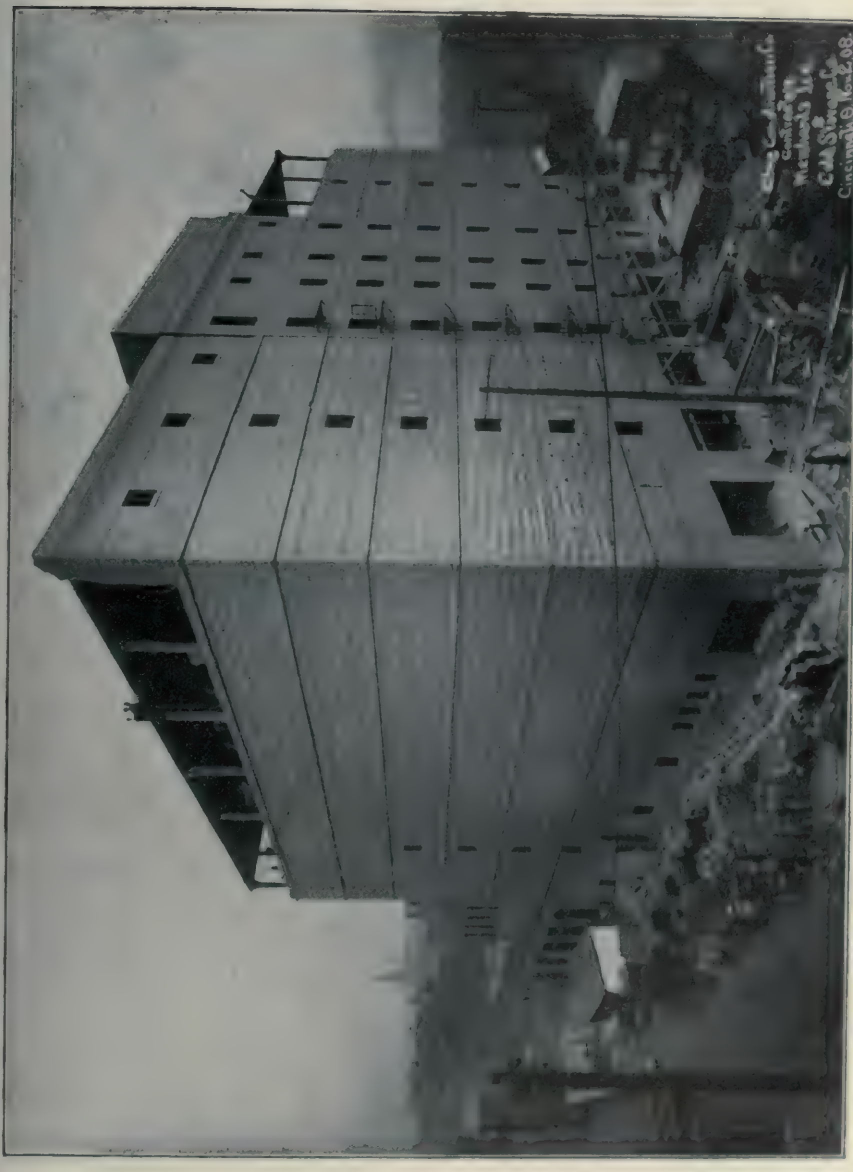
In these structures rooms are frequently kept at widely different temperatures for the preservation of goods of various characters, subjecting the material in the floors to severe temperature stresses. These are best resisted by multiple way reinforcement such as that presented by type IV, figure 4. This type of construction has a further advantage for this class of buildings in that the insulation is very readily applied to the concrete work as follows:

The sheets of cork or composite insulation are placed on the flat centering and nails driven into them or up through them, sticking above their surface, and which become rigidly fixed in the concrete as soon as it is set. The concrete is deposited or poured on to the insulating material.

In this class of buildings it may be noted that there is a large saving in the expense of insulation, since the additional amount of insulating material which would be required to cover beams is eliminated with all of the trouble and difficulty incident thereto.

The columns for convenience in insulation should have a plain conical capital instead of moulded capital ordinarily used.


MERCHANTS ICE
COLD STORAGE CO
CINCINNATI O.



This part of the work may then be insulated very readily indeed.

We show in the accompanying cuts, pages 256-257, views of the big Merchants Ice and Cold Storage building at Cincinnati, Ohio, put up of this type and insulated in this manner. An interesting trade pamphlet, showing details of the insulation in this building, is published by the Armstrong Cork Company, of Pittsburg, Pa.

ARTICLE 5.

Packing Plants.

The requirements of the packing plant are somewhat similar to the cold storage building. A good example is that of the J. T. McMillan Company building in St. Paul, using the same flat slab type and details of the insulation of the cold storage rooms being similar to those employed in the Merchants Ice and Cold Storage building. Quite an advantage was found by the owners in the fact that the flat slab type enabled them to place a large number of inserts so that tracks and runways could be readily attached for handling their product without loss of height through beams and the interference of ribs with the placing of supports for the conveyors.

ARTICLE 6.

Banks, Jails, Hotels, Etc.

Taking up now the third class, banks and jails: The bank is ordinarily fitted with vaults which are intended to keep the burglar out, while the jail is intended to keep the criminal in, and for this purpose no more suitable material than reinforced concrete can be found. For a vault, walls two feet thick, reinforced with four layers of chrome steel bars crossing each other at angles forming a small mesh at different planes through this thickness combined with a concrete made rich and with the hardest aggregate, flint stone preferred, that can be found, will give the most ingenious burglar a difficult job to drill or blast through.

For the jail, reinforced concrete is an excellent material, thoroughly sanitary and much harder to cut or dig through than any other type of construction.

A jail delivery where the building is reinforced concrete fit-

ted with suitable steel doors would be a far more difficult matter than with any other type of construction put up today.

Reinforced concrete for hotels, apartment houses or residences is desirable from the standpoint of sound-proof properties, from the sanitary standpoint as well as from the fireproof standpoint.

ARTICLE 7.

Concrete Steel Construction for Office Buildings.

In the larger cities where land is of enormous value, office buildings are constructed to a great height, requiring heavy and expensive steel frames if put up in the old-fashioned manner. For this class of building reinforced concrete would effect a very substantial saving indeed where properly and economically designed.

The first concrete steel sky scraper erected was the Ingalls building of Cincinnati, a seventeen-story structure built on the Ransome system and in which all engaged in concrete steel construction take no little pride, as this structure is indeed a credit not only to the eminent pioneer, Mr. Ransome, but also to the class of construction in general.

Why, with such an early precedent, building ordinances in not a few of our large cities are endeavoring to limit the height of structures which may be put up of this material, except by some limitation that is reasonable in the way of working stresses upon the material, is one for which no excuse can be offered from the engineering standpoint.

We show in the accompanying cut, figs. F and G, the twelve-story mercantile building constructed for M. Born in Chicago, the well known firm of Holabird & Roche being the architects. The capacity of this building is two hundred pounds per foot of floor. The work was erected rapidly and without accident.

It is evident that if it is safe for two hundred and fifty pounds per foot the same section of columns could be used for the ordinary office building had the height been increased to nearly double the number of stories.

In New York, those responsible for the building ordinances seem decidedly adverse to concrete construction. One of the ordinances at present proposed for the new code is a limitation of the thickness of the slabs to not less than ¾″ per foot of

Fig. F. Mercantile building for M. Born, Chicago, Ill. Holabird & Roche, architects. Alling Construction Co., builders. Mushroom System.

Fig. G. Mercantile building for M. Born completed.

span, not counting that part of the material which is used for fireproofing the construction.

Looking into this proposed ordinance we find that for an ordinary span slab of 26′ 0″ there would be required a thickness of slab in the neighborhood of 21″, a dimension ample for a working load of 3,000 pounds per square foot over the full area without excessive working stresses, providing the reinforcement was put in in proportion to the concrete required.

As this is a little matter of thirty or thirty-five times the capacity required for the ordinary hotel or office building we naturally question whether this astonishing degree of ignorance of the characteristics of concrete is quite as innocent as it appears on the surface. In other words, the question is logical whether this apparent stupidity is intended merely as a cloak to utilize the so-called department of public safety, as a unique, legally fireproof department of public graft, to be operated for the joint benefit of those politically higher up and those commercially interested in some old fashioned types of fireproof construction.

American Type Founders' building.

CHAPTER XXI.

Artistic and Commercially Practicable Concrete Surface Finishes.

ARTICLE I.

Stipple Coat.

We find that applying a stipple coat, either rough or smooth as desired, a very pleasing effect can be readily obtained. An example of this treatment is shown in the figure of the Smythe block, Wichita Kansas. The writer has adopted this finish for most of his bridge work, as it gives a greater appearance of strength, readily covers up the minor imperfections in the centering, and affords a pleasing contrast with the highly ornamental stone railings with which he prefers to finish his work.

The stipple coat is usually applied with a broom corn brush and consists of a thoroughly mixed grout of a neat cement one part, and one part sand. Treatment of the surface should be as follows:

Wet down the face of the wall thoroughly with a hose. Then apply the stipple coat, spattering it on. This method of treatment is being adopted to a large extent by architects in the finishing of exterior cement plaster walls for residences. A very neat effect indeed is obtained in this manner at a low cost. Expanded metal wire lath is nailed to the studs, plastered with a Portland cement mortar with generally ten per cent or such a matter of hydrated lime, the mixture being practically one cement to one and one-half sand and finished with a stipple coat as outlined.

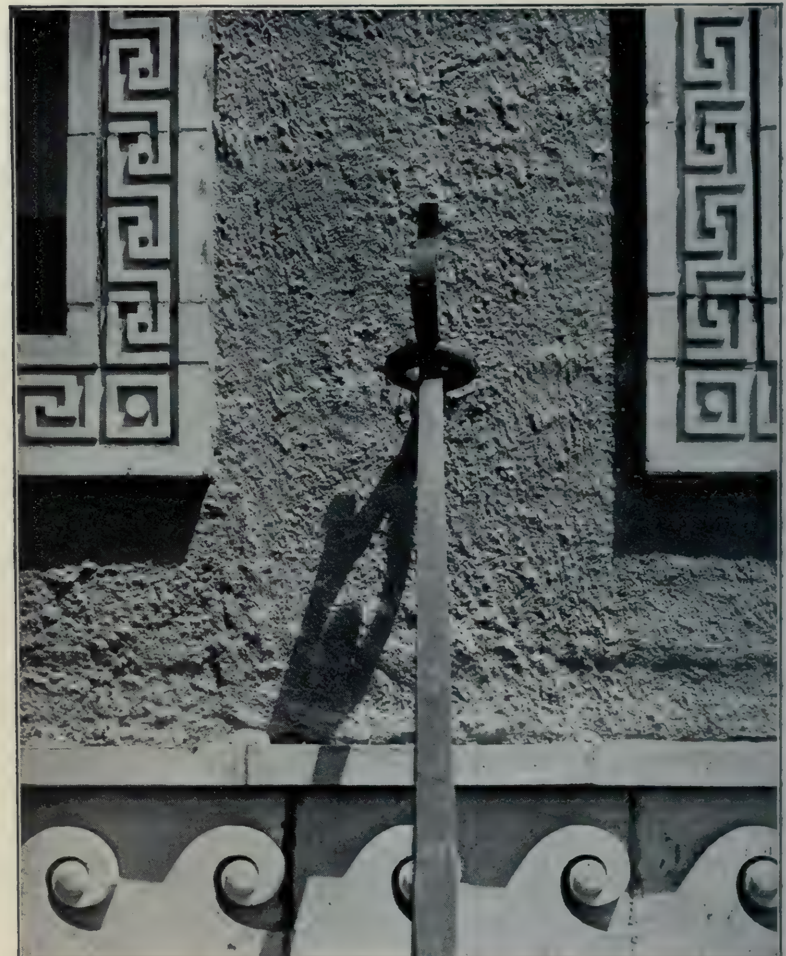
Failure of cement plastered walls may be attributed in the main to failure of the contractor to use enough cement. The general argument of too many workmen is that the concrete will not be good if it is made too rich, while as a matter of fact it requires a rich, strong mixture to withstand the frost and severe climate in all northern states. Plaster work which would stand without injury in Cuba and Arizona would go to pieces in short order in Minnesota or Manitoba. A properly applied cement

coat of a rich mortar, however, will stand the climatic conditions in the north while in the south also a rich mixture is to be decidedly preferred.

ARTICLE 2.

Plaster Coat on Rough Cast Concrete.

A very expensive effort was made to secure a good surface finish on the Grand avenue viaduct in Milwaukee. The specifications required that the inside of the forms be lathed with ex-



Stipple finish of Smythe Block, Wichita, Kas. Louis Curtis, architect.

panded metal and plastered with a mixture of plaster of Paris and lime. This plaster coat was to be oiled in advance of placing the concrete and the concrete to be placed and tamped in layers. On removal of the forms, notwithstanding the greatest care on the part of the contractor, the line of demarkation between the several layers was plainly visible and it was found impossible to put up the work without blemish as required in the specifications.

The work was finally carried out by removing the forms on the exposed surface as early as practicable and plastering with a thin coat of rich cement grout. This is a practice which the writer does not recommend, as wherever the mass of the concrete has had time to get fairly hard this plaster coat is liable to check and scale off, though occasionally it has been applied with a fair degree of success before the concrete has had time to thoroughly harden.

ARTICLE 3.

Finish Obtained by Brushing and Washing.

Mr. Henry H. Quinby of Philadelphia appears to have been one of the first to introduce a method of brushing and washing the concrete surfaces, bringing into relief the aggregate used. The process consists of removing the forms after the material has set, but while it is still friable, and then immediately washing and rinsing the cement which has formed against the mould and thereby expose the particles of sand and stone. The appearance then depends upon the character of the aggregate in the concrete its color and the uniformity of its distribution in the mixture. The time to be allowed for setting before washing must depend upon the nature of the cement and the temperature conditions. Quick setting cement and warm weather call for the removal of the forms from seven to ten hours. The appearance may be controlled somewhat by the extent of washing which may be to the extent of leaving the stone aggregate in decided relief producing a rough coarse texture much admired by the majority of architects.

An interesting article on this subject will be found in a book entitled "Concrete Factories," by Leslie, published by Bruce and

Banning of New York, and in some of the older numbers of the "Cement Age."

A well written paper on the same subject has been published by the Universal Portland Cement Company in their trade bulletins, numbers 54, 55 and 56, which, through the courtesy of the company, is reproduced in part herewith:

The ordinary concrete surface, it must be admitted, is anything but pleasing in appearance, being usually a comparatively smooth, lifeless surface of a somber grayish color. It makes but little difference what cement, sand or aggregate is used, or in what proportions they are mixed, the general aspect of the unfinished form surface is the same. There may be the greatest difference in color, shade and texture of the aggregate used in two separate concrete surfaces, yet unless they are so treated as to bring out and expose the aggregate, the resulting surfaces will look alike.

It is quite difficult to distinguish an ordinary unfinished concrete surface in which bank gravel is the aggregate from one in which crushed red granite is used, but the same surfaces, if subjected to any one of a number of different methods of surface treatment, will present a marked and pleasing contrast in appearance. It is the monotonous sameness in the appearance of concrete work that architects object to so strongly. To show what can be accomplished in producing pleasing, artistic and commercially practicable surface finishes for concrete work is the object of this article.

On the opposite page are three photographic reproductions of brushed concrete surfaces. The difference between these surfaces and that of ordinary gravel concrete is very striking, yet they are all practical, commercial finishes, and can be obtained by the use of material from ordinary gravel bank.

Figure I shows a comparatively fine, even-grained surface, composed of one part Portland cement and three parts of fine sand, all of which pased a No. 8 and was retained upon a No. 50 mesh screen. Figure II is very much like Figure I in general appearance and color, but of a rougher, more uneven texture. This surface is a 1:3 mixture, with coarse sand, passing through a No. 4 and retained on a No. 8 screen. Figure III represents a finish made from a 1:3 mixture of cement, and ¼″ to ½″ pebbles. Thus these surfaces are identical in every respect, ex-

Finished Surfaces.

Figure I

Figure II

Figure III.

Surfaces reduced one-half of originals.

Finished Surfaces.

Figure IV.

Figure V.

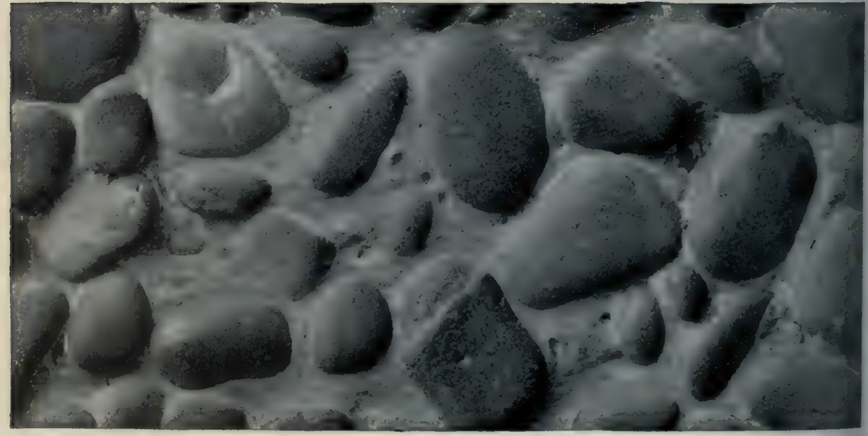


Figure VI.

Reproductions are actual size.

cept as to size of aggregate. The three surface finishes were all produced by the same method of treatment.

The cuts give but a poor idea of the appearance of the actual surfaces, as the color texture which give life and individuality to any surface are lacking. To appreciate the value of this finish for concrete work, the surfaces from which these cuts were made should be seen.

Figures IV, V and VI are three cuts, photographic reproductions of concrete surfaces similar as to surface treatment to those shown, but differing from them in the aggregates used.

Figure IV shows a decidedly pleasing, even grained surface composed of one part Portland cement and two and one-half parts red granite screenings, all of which passed a No. 8 and was retained on a No. 16 sieve. Figure V is a reproduction of a surface composed of one part cement to two and one-half parts ordinary, quarter inch, granite screenings, the material passing a No. 8 sieve being rejected. Both these surfaces are quite similar in every respect in texture, that represented by Figure No. V being of a rougher texture than the other. As the cement is barely perceptible on these surfaces both look very much like rough, undressed red granite, the color being practically the same as that of the screenings of which they were made. Figure VI represents a treated surface composed of one part cement to two and one-half parts of black pebbles, varying in size from those retained on a No. 10 sieve to those passing a 1¼″ mesh. The cut gives but a poor idea of the pleasing contrast between the light colored cement background and the black pebbles which stand out in bold relief from the surface.

Comparing these cuts and those in the preceding page, quite a difference in general aspect and texture is to be noted, and an examination of the actual surfaces would reveal a still greater difference in appearance due to the striking variation in color and size of the aggregate used. Had these six surfaces been left untreated, from surfaces they would have looked practically identical.

By varying the kind, size and proportions of the aggregates, surface finishes of practically any desired color and texture can be obtained, the possibilities being limited only by the number of different aggregates available and the combinations of same. A great variety of finishes may be produced by using red and

black granite and limestone screenings, black and white marble chips and different colored pebbles and sands. But as the photographic reproductions do not reveal the colors, Figures IV and V, while reproductions of surfaces in which red granite was used and Figures Nos. I, II, III, and VI for surfaces finished with various colored pebbles and sands.

All the cuts shown represent brushed concrete surfaces, the process consisting of simply brushing the surfaces with a stiff brush, permitting it to harden for a few days and then treating it with a dilute solution of hydrochoric acid, the method of procedure being as follows:

Having decided upon the general color scheme and texture of the desired surface the first step is the making and treating of small sample surfaces. A limited amount of experimenting with the materials available will always prove profitable. The color and texture of the finished surface depends upon the color, size and proportions of the aggregates used, and the successful reproduction of the desired surface is dependent upon the proper selecting, grading, proportioning and mixing of the materials and the proper placing and finishing of the surface. Upon determining by experimenting the proper size and proportions of aggregates to produce the desired effects and the proper consistency of the mix, adhere strictly to them; that is, take the trouble to measure the materials for each batch of concrete and to gauge them with a measured amount of water. The results obtained will more than justify the extra expense this will entail over the all too prevalent method of measuring material by wheelbarrow loads and adding the water with a hose; in fact, uniform results cannot be obtained unless the work is done as pointed out.

The slightest imperfections and irregularities in form surface are transferred to the concrete, producing unsightly surfaces when left untreated, and a pleasing surface cannot be obtained by a nicety of form construction alone. For brushed surfaces, all that is required of the forms is that the face lagging be kept true to surface and the joints be tight. For surfaces too large to concrete in one day the forms should be so constructed as to permit of the removal of sections of the face form. This can be accomplished by setting the studs or uprights back a few inches from the face lagging and connecting both by means of cleats

and wedges. The face forms also should be well oiled to prevent the concrete sticking to the forms. In large areas the introduction of buttresses and panels or the breaking up of the surface by horizontal joints or courses will add greatly to the appearance, the joints being simply indentations in the surface produced by beveled beads fastened to the forms. It is extremely hard to join two different days' work so that the joint is not perceptible and unsightly, and the breaking up of the surface as indicated will greatly assist in the concreting if care is taken to end and start succeeding days' work at a course or joint.

The facing material should be from one to one-and-a-half inches thick, the remaining thickness of the work being composed of ordinary concrete, but the facing and backing must be deposited at the same time so as to make one solid mass, thereby insuring perfect bond. The facing material may be applied to the forms just ahead of the backing, which is placed against and rammed into it, or the backing first and then brushed back from the form with a spade and the facing material deposited between the backing and the form. Both these methods have been successfully used. A third and possibly the best method of placing the facing material consists of the use of what might be called a metal facing form or mold, constructed and used as follows: To short lengths of 3-16″ iron plates 8 or 10 inches wide and 6 feet long, three 1 or 1½″ angles are riveted, placing an angle at the center of the plate and one about six inches from each end. One edge of the plate should be slightly flared to assist in depositing the material and this edge provided with handles. The metal facing plate is placed against the wall form with the handles up and the angles tight against the form. The space between it and the back of the wall filled with the concrete backing and the 1 or 1½″ space between the metal form and the face form filled with the facing material. The metal form is drawn almost out, and after thoroughly tamping the backing against the facing the process is repeated.

For brushed surfaces the forms must be removed from the work as soon as possible and the concrete surface brushed while still green. It is not possible to state how old the work should be before removing the forms and brushing the surface. This will depend upon a number of conditions, the character of the work, cement and aggregate used, consistency of the mixture,

and very much upon the weather conditions. As a rule in hot weather the forms can be removed the next day and the surface brushed, but in cold weather the facing form cannot be removed so soon, several days perhaps a week being required for the concrete to attain the necessary hardness and strength. Care must be taken that the brushing is not done too soon, as little particles of aggregate will be removed, resulting in a pitted, unsightly surface. On the other hand the longer the surface stands before being brushed the more brushing it will require to remove the film of material that has flushed to the surface. Brushing should be done just as soon as it can be without removing particles of aggregate. When this can be done can only be determined by experimenting with the particular surface. An ordinary scrubbing brush with stiff palmetto fibers or a metal wire brush will answer for the work. Two or three days after the brushing the surface should be washed down with a dilute solution of commercial hydrochloric acid, one part acid to two or three parts water. The acid should be applied with an ordinary calcimining brush and the walls thoroughly rubbed, while wet with the acid, with a stiff vegetable fiber brush. The acid should not be allowed to remain on the surface for any length of time—not over half an hour—and should be washed off with a hose and clean water. It is important that the surface be thoroughly washed after the acid treatment, for if it is not it will have a mottled, streaky appearance.

A desirable surface can be obtained by simply brushing and then washing with a hose and clean water, but the final acid treatment in connection with the brushing will produce a still better surface.

This method of treatment removes the film of mortar that has flushed to the surface, exposes the aggregate, erases all traces of form markings and produces a rougher, more artistic surface. The roughness of the surface breaks up the light, the color of the aggregate adds variety and life, and we have a pleasing, artistic, true concrete surface.

ARTICLE 4.

Finish by Tooling.

Where the architect is not limited in the point of cost, an excellent effect can be secured by tooling the surface of the con-

Bridge at Stetten.

Exhibit of cast stone.

The railing of bridge at Fergus Falls, Minn. John Lauritzen, Contractor. C. A. P. Turner, Engineer. Railing manufactured by the National Stone Mfg. Co., of Minneapolis.

crete either by hand or using pneumatic tools. The effect will depend largely on the character of the aggregate and where this has been carefully selected the finish is quite attractive, especially when the surface is broken into blocks by rustication or grooves.

The expense of tooling ranges from five to ten cents per surface foot, dependent on the equipment used, while that of brushing and washing should not run more than one-fifth of this amount.

ARTICLE 5

Cast Stone.

Where suitable aggregate is available an excellent building material is made by casting a concrete in sand moulds.

The process is similar to the iron moulders' art. Wood or plaster patterns are used, a sand mould prepared and the concrete cast mixed to about the consistency of cream. When the resulting material has been tooled it is hard to distinguish it from the natural stone.

In cost it cannot be manufactured to compete with the natural stone where there is little freight to pay, but where the work is at all complicated and there is a duplication of the parts and quarries of good building stone are not situated convenient to the locality, there is a good field for this product.

It has been very successfully manufactured in Toronto, St. Louis, and other parts of the country and also in Germany.

The accompanying cuts show, first, example of the cast stone work in a German bridge at Stetten; second shows a small exhibit of cast stone of the Northwest Cement Products Association, while the third is a cut showing a railing of cast stone on a bridge at Fergus Falls, Minn.

CHAPTER XXII.

Casualties and Accidents in Concrete-Steel Construction Compared with Those Occurring with Other Types of Building Work.

ARTICLE 1.

Definite Responsibility.

The introduction of concrete-steel construction has been very rapid indeed, and as a new line of building, such accidents as have occurred in its installation have attracted undue attention, as is invariably the case with any class of accident which involves a feeling that the danger is new and unknown. This feeling has been emphasized to no small extent, due to the fact that the engineer and architect have found such a ready means of shirking that degree of responsibility which belongs rightfully to them and charging all accidents from whatever source to careless, inferior or indifferent workmanship. The poor laborer in the concrete gang, not having the ear of the public, does not talk back and properly present, as a rule, his side of the case.

The designer comes along with figures showing beyond question that he has properly provided for the bending moment in a beam and considers that he has cleared himself beyond question. As a rule, proof of this kind furnished by those responsible for the design of structures which have failed, contains no reference as to how well they have designed the connections or how thoroughly they have tied the construction together in a monolithic manner. No reference is made as to whether they have placed the ties in a column so closely together that it requires more than ordinary care to properly place the concrete. In other words, their defense contains no reference, as a rule, to the *really vital points* of construction.

Perhaps no small degree of indirect responsibility for such bad accidents as have occurred in concrete-steel construction is chargeable directly to the class of literature which passes muster as scientific, with reference to concrete-steel construction.

Take, for example, one of the latest works on reinforced concrete construction by Turneaure and Maurer. We find in this work illustrated designs of columns which are among the most objectionable and dangerous that can be devised and yet no mention is made of the dangerous characteristics of the detail illustrated and as far as careful perusal of the work would indicate this has not been recognized by the authors.

It is easy enough to find a small area of rough concrete surface where on the whole the workmanship has been good; it is easy enough to state where a collapse has occurred that had the centering been left in for a considerable additional length of time that the work would have stood, and there is a large degree of truth in this assertion, and yet there is an equal degree of truth in the statement that had the design been conservative the work would have stood even in its partially hardened condition, or, at the worst, would merely have deflected somewhat from the desired form without coming down by the run and injuring those under it.

The idea that there is something vague, unknown and mysterious in the failure of a concrete structure is well illustrated by the editorial in the Engineering News regarding the Bixby Hotel failure, in the following words:

"The work whose character is impugned lies in a rubbish heap, whence it is impossible to extract supporting evidence."

This statement, coming as it does, from the editors of the first engineering paper in the country, can only be accounted for on the ground of nearly complete lack of familiarity with the business.

Compare the treatment of the failure of the Quebec bridge by the brilliant editorial staff of this paper with that accorded the failure of the Bixby hotel; in the first instance they pointed out almost immediately the true cause of the failure in a clean cut, concise manner; as regards the Bixby hotel, they have neither a word nor apparently a critical opinion as to the character and make-up of the design. We can only account for this lack of well formed ideas and clean cut opinions on the part of this editorial staff on the ground of lack of practical familiarity with this type of construction.

When the editors of such progressive engineering papers as this fail to point out engineering defects in their usual fearless

manner, it is fair evidence that many of the profession are laboring in the dark.

That the architectural profession are even farther behind than the rank and file of the engineering profession is evidenced by the editorial presented in so many of the architectural papers a year or so ago to the effect that the time of removal of the forms from reinforced concrete work was indeed a proper time for prayer. The writer is sorry to admit that his early religious training fails to throw any possible light on what connection there might be between prayer and the conditions favorable to the hardening of Portland concrete.

In the Engineering News editorial referred to the statement was made that the gray of our concrete structure is already too much stained with blood. That there have been unfortunate blunders and loss of life in concrete construction the writer does not mean to gainsay. On the other hand, to infer there is not the same chance for negligence and loss of life in the executing of work with other types of construction is a point which reference to the records of the deaths and accidents in structural steel construction and concrete construction will refute.

These will show that with all the ignorance and incompetence that has been unquestionably displayed in the introduction of reinforced concrete the total number of deaths and accidents are far less than those which ordinarily occur in the erection of an equivalent amount of structural steel construction

ARTICLE 2.

Safety to Occupant.

As far as the general public is concerned, concrete steel construction possesses an element of safety not found in any other type. Outside of such accidents as have occurred during the erection of the work, it is hard to find a single record of failure of a concrete building that has stood for a period of six months after the work was placed, for the evident reason that the concrete is increasing in strength as the process of hardening goes on for months and years, while with others types of construction the materials are deteriorating rather than improving.

For instance, we frequently hear of a collapse of a timber

and sometimes of a structural iron and steel building which is occupied. Quite a few examples of this class of failures can be found in the back numbers of the engineering papers of the country, while as far as the writer knows, none can be found with the concrete structure, no matter how badly it may have been originally designed.

ARTICLE 3.

Records of Accident with Concrete and with Structural Steel Compared.

Returning now to the question of number of fatal accidents which have occurred in the erection of concrete steel construction work, as compared with those which are occurring right along in the erection of structural steel construction, we may make the following general statement: that at the present time there is more reinforced concrete building work under construction than steel building work; that the records of the trade unions show in the last two years from eight to twelve deaths of structural iron workers per month, slipping and falling from beams, crushed in handling or placing girders, and so on down the line. These records may be readily verified by referring to the Bridgemen's Magazine.

In the year of the great engineering disaster at Quebec, loss of life amounted to over twenty per month, among union iron erectors, and to probably twenty-five to twenty-eight per month, counting union and non-union.

In the writer's personal experience in some twenty odd years in structural steel construction there have been nearly a dozen accidents in which men were killed or injured in putting up structural steel construction through no fault of the designer or neglect on his part of anything which he could foresee and obviate were he to do the same work again.

In putting up something like six hundred acres of floor in concrete steel buildings there has been not a single accident which is directly chargeable to the risk of construction. One fatality occurred on the John Deere building, at Omaha, which appears to have resulted, so far as known, from the workman becoming dizzy, fainting perhaps from heat and falling off the the edge of a floor nearly an acre in extent.

In general, we may say that the advantage from the standpoint of safety in concrete steel construction is that the men invariably have a solid floor to work upon; that they are not required to work aloft with nothing but a grid of steel beams to work upon, which are frequently slippery from frost, and from which a fall is serious and generally fatal.

The great risk in the business of concrete-steel construction is that due to engineering ignorance and incompetence, which will rapidly disappear as the character of the construction becomes better known and the responsibility for failures is placed squarely where it generally belongs, on the shoulders of the designer of this class of construction.

In fact, the writer's strong preference as an engineer for concrete steel construction as against structural steel work is on the ground that it is safer for the men to put up and that he can feel satisfied that by the exercise of ordinary care there is no excuse for accident or injury to the men in the erection of this class of work, while structural steel construction is a type of work distinctly hazardous and in which accidents are likely to occur occasionally with those who, as a rule, exercise good judgment and the greatest care.

ARTICLE 4.

"Thank You" Engineering Advice.

The writer has at times given engineering advice to fellow workers in the field of building construction without charge. In general, such advice is given weight in accordance with the manner in which it has been obtained. In other words, cost nothing, worth nothing.

As an example of this, a friend of the writer, an unusually bright sort of fellow, lacking only the balance wheel and safety valve of a thorough technical training to place him among the first in his line, submitted to the writer a bold and daring type of construction, which the writer reported on, and cautioned him particularly as to the limits which he considered it would be conservative to place on its use. This advice, costing him nothing, was given little attention, until a failure resulted, costing him from six to eight thousand dollars, which he could ill afford to lose in his business. Had the writer charged his ac-

quaintance a consulting fee of two hundred or such a matter for his advice regarding the type of construction, perhaps it would have received attention in accordance with the cost of the report. That the work was brought to the writer's office to redesign, in no wise relieved the writer from feeling a certain sense of indirect responsibility in having failed to charge a sufficient amount for the report to make it appear of value sufficient to be followed by the recipient.

In fixing the price of this treatise on concrete steel construction, the writer trusts that it is sufficiently high that those who may purchase it will feel compelled, in order to get value received, to read it through instead of placing it on the shelf as a typical dry, tiresome and uninteresting technical disquisition on the subject—even though it may be worse than the average in this respect.

APPENDIX

PART I.

Suggestions for a Concrete Constructor's Library.

ARTICLE 1.

Theoretical Treatises.

The contractor or builder in the line of reinforced concrete construction desires to have a thorough knowledge of such literature as is published on the subject. This information is of double value in discussing questions which may be brought up by the engineer and architect who have taken the time to read more widely than the practical constructor can find time ordinarily to do, hence a condensed list of such works as will present the diverging views of various authors form a valuable library for the practical constructor. These noted will prove of value, not necessarily altogether from the standpoint of the use they may be to him in teaching him methods of figuring and executing the work and the character of details used, as to give him a fair idea as to the views which certain authors hold by comparing the views and opinions of one with another in the light of his own practical experience, he may be able to uphold his own views based on that experience as against the opinions which may be brought forward by the engineer or architect student of the art.

A number of these books have been reviewed in a pamphlet entitled "Literature on Reinforced Concrete," compiled by the Engineering News Book Department, containing a reprint of an article in the Engineering Digest by Leon S. Moisseiff, which goes into the subject more at length than space here permits and from an entirely different standpoint.

If the contractor desires information bearing upon the question of deflections and test loads which he is forced to guarantee he will probably find the present treatise the only one in

the field which will enable him to get anywhere with certainty in computations of this character when dealing with the natural types of concrete steel construction.

The first treatise to which we will call attention is "Reinforced Concrete," by Charles F. Marsh. This is one of the earlier pioneer treatises on the subject. The scope of information obtained is valuable from the historical standpoint, showing the evolution of various types of construction and the general theory of reinforced concrete is well presented from the standpoint of the college professor, rather than from that of the practical constructor.

Reinforced Concrete. By A. Considere, translated from French by Leon S. Moisseiff. This work by the eminent inventor of the best and most practical type of reinforced concrete column under construction should be in the library of every practical constructor. The price is moderate—$2.00 net.

Concrete and Reinforced Concrete Construction. By Homer A. Reid, follows along very similar lines to the work by Marsh as far as theoretical treatment is concerned, though containing much supplementary matter of considerable practical value.

Principles of Reinforced Concrete Construction. By Prof. Turneaure and E. R. Maurer. The authors suggest that their purpose was to cover in a systematic manner those principles of mechanics underlying the design of reinforced concrete: to present the result of all available tests that may aid in establishing coefficients and working stresses and to give such illustrating material and actual designs as may be needed to make clear the principles involved. The University of Wisconsin has done considerable work in testing simple beams, which, with the theory of simple beams, is well treated by them. Like the preceding treatises they have discussed the natural types of construction in an erroneous and very imperfect manner. The theory of arches is fairly well presented.

Reinforced Concrete. By Buel and Hill. The theoretical treatment is somewhat similar to treatises previously discussed, more attention being paid to arches than building work.

Engineer's Pocket Book on Reinforced Concrete. By

E. Lee Heidenreich. One of the most satisfactory books of its class and may well be placed in the library of the constructor.

CEMENT AND CONCRETE. By Lewis C. Sabin. A book distinctly dealing with the material cement, rather than its use in the construction, but is an admirable treatise on concrete as a material, well meriting a place in the constructor's library.

REINFORCED CONCRETE IN FACTORY CONSTRUCTION. By the Atlas Portland Cement Company and distributed by this company as trade literature, and is worthy of the time spent in reading.

EXPERIMENTAL RESEARCHES ON THE CONSTITUTION OF HYDRAULIC MORTARS. By Henri Le Chatelier, translated from the original by Joseph L. Mack. Contains much that is interesting to the cement worker, particularly those who are dealing with artificial cement stone.

ARTICLE 2.

Flat Plate Theories.

The constructor who is mathematically inclined is interested in the flat plate theories, since they present to the mathematician the clearest and most logical explanation of the phenomena which he observes with the natural or distinctive concrete types of construction. These theories may be summarized as follows by authors:

"Grashof's Flat Plate Theory," see "Lanza's Applied Mechanics."

"H. T. Eddy's Flat Plate Theory," see year book of the University of Minnesota, 1897-1901.

"Merriman's Flat Plate Theory," see "Mechanics of Materials," by Mansfield Merriman.

"Bach, Elastizitat und Festigkeit," von C. Bach.

ARTICLE 3.

Bulletins Descriptive of Tests.

On the subject of tests, those by the Universities of Wisconsin and Illinois are of value. Bulletin 8, 1906, test of shear and bond, University of Illinois; bulletins 12, 14 and 19, tests

of reinforced concrete beams; bulletin 20, tests of columns, although not of the vertically reinforced and hooped type are of value. Tests of plain or reinforced concrete beams by Withey, University of Wisconsin, the constructor will find of interest; also Bulletin 344, tests of beams U. S. Geological Survey.

ARTICLE 4.

Literature Regarding Damage to Tile and Concrete Construction by Fire.

The contractor in the line of reinforced concrete is placed in competition with the burned clay advocate. The stock argument of the advocate of tile fireproofing is that a small specimen of burned clay will withstand high temperatures in a furnace, while a small sample of concrete would, under similar conditions, be disintegrated by continued heat of the furnace. For the reasons pointed out under "Fireproof Properties of Concrete," this comparison is no indication of the deportment of two classes of material in actual conflagration.

Examples of the failure of tile are needed by the reinforced concrete constructor to meet this plausible but misleading line of talk. The damage to hollow tile by a fire in the Horne building at Pittsburg is well described and illustrated in the Engineering News, vol. 37, p. 313, May 20, 1907. The Parker building, New York City, is another excellent example, which was immediately seized upon and advertised by the publicity bureau of the burned clay interests in the daily papers of the country as the latest example of the failure of reinforced concrete, while as a matter of fact, there was no reinforced concrete in the building except that in the form of cement mortar which was used between the flat hollow tile arches. The nearly complete destruction of this building is well covered in the report of the New York Fire Underwriters, April 22, 1908. In the Baltimore fire the damage to tile and such concrete buildings as came in its path is well described in the Engineering News, vol. 51, pp. 145, 169, 194, 200, 261, 276, 528. The San Francisco Earthquake and Fire, 1906, see book by Roebling Construction Company; also Engineering News, vol. 55, pp. 478, 509, 521, 548, 580, 584, 607, 622, 694, 707; vol. 56, pp. 136, 333, 409, 474.

The constructor should include in his library the "Rules and Practice of the United States Patent Office," which may be

obtained by simply sending his name and address to the Commissioner of Patents.

ARTICLE 5.

Trade Literature, Catalogues, Etc.

He should also keep up to date with all trade literature in this line. This is desirable for two reasons:

First, that he may be familiar with everything on the market;

Second, that by means of this familiarity he can form a clean-cut idea as to the relative economy of the various systems and know how much chance there is to figure additional margin of profit if he knows who his competitors are, and the system on which they are figuring.

The contractor is not in business for his health, and the profit that he can add to his estimated cost is a matter of judgment which must be based on his estimate of the ability and judgment of his competitors and the cost of the type of construction they propose to figure upon in competing with him.

If the constructor can add twenty-five or thirty per cent profit in a clean competition against those less skilled in the art instead of the ordinary ten or twelve per cent, he is entitled to it and is foolish if he does not make use of his legitimate opportunity.

PART II.

Some Facts Regarding the Shipment of Cement Which May Prove of Value to the Contractor.

In purchasing cement for a piece of work the contractor usually specifies that it shall meet the requirements of standard specifications. Suppose he is doing business in the Dakotas or in Manitoba, etc., and the cement is received by way of the lakes. Representative of the owner or architect samples some of the cement, takes a sample out of one bag and reports that it does not come up to the specifications.

It is well for the contractor before jumping to conclusions and accepting the condemnation to examine the sacks to see whether they have been broken and sample them himself and check the test.

It too frequently happens that in shipping cement by water that the boat company breaks open a few sacks in handling and as they are responsible for the delivery of the number of sacks called for in the consignment they sweep up the deck, dump in the sweepings into the sacks which have been broken open and tie them up with a new string and get credit for the proper number of sacks delivered. The sacks will quite likely be tied up with a piece of tarred Manila instead of cotton string and the character of the material will be evident by the dirt, sticks, straws and the like scattered through it.

Lack of familiarity with what may happen in this wise several years ago cost the writer about $2,000 in changing brands of cement and paying the advance in price on the same when covered by contract with a dealer at the lower rate on the first brand.

After one has paid sixty thousand prices for a little two cent bit of information of this character he begins to appreciate its value in the commercial field.

PART III.

Some Facts Regarding Patents and Patent Laws of Interest to the Owner, Builder, Architect and Engineer.

ARTICLE 1.

Reason for Patent Protection.

Patents are granted to the inventor as a method of encouraging advancement and development of the mechanical arts. The intent is to protect the inventor by giving him the sole right for a limited period to use, make and sell the product of his inventive genius. To the contractor this is important if he has originated some new labor-saving device or type of construction. The points to be borne in mind in securing this protection are as follows:

ARTICLE 2.

Time to Apply for Patent.

First, the application for the patent must be made before the invention has been publicly used for the period of two years.

ARTICLE 3.

Liability of Owner.

Second, no suit can be maintained prior to the issue of the patent, but after the patent has issued suit may be maintained against structures built prior to issuance of patent.

This phase of the case makes it incumbent on the contractor or owner to exercise care in the use of similar types of construction to those which he knows to have been first in the field even though some years have elapsed since they were first used and no patents have apparently issued.

This reason is evident since any invention of value the first inventor in the field desires, and is entitled to make his claims as broad as the state of the art at that period allows, and if his attorneys are sufficiently skilled in drawing claims he may cover his type so fully as to collect royalty on all similar types for the

life of the patent when issued. Protection of this complete character requires the highest degree of skill on the part of the attorney and may require a long drawn out argument extending through a period of a number of years with the examiner of the patent office.

ARTICLE 4.

Advantage and Disadvantage to Inventor by Delay of Issue.

While the inventor is at a disadvantage due to this delay his compensation lies in the fact that the limitation of his monopoly dates seventeen years from issue and not from the date of application for the patent. Thus he may readily secure more than his seventeen years monopoly, providing there is no fraud in his method of procedure and he has been acting in good faith in prosecuting his application.

ARTICLE 5.

Impossibility of Securing Advance Information as to Scope of Patent.

In the examiner's office the files are not open to outside inspection, hence it is impossible to know the exact extent of protection that the first inventor in a given field has secured until after the final issue of the case, and the longer this issue is delayed the stronger as a rule will be the patent when finally issued, since it has probably passed the closest scrutiny in the office.

In case, however, foreign patents are granted before the United States issue some idea as to the scope of the United States case may be gleaned from the foreign patents issued.

ARTICLE 6.

Disadvantage of Secrecy Regarding Invention.

Secrecy regarding an invention is not always the best policy unless a record is made in the form of a caveat while protecting an invention. Frank disclosure to a number of friends as witnesses often serves the inventor in an attempt to prove later his priority over some other applicant.

At present in the United States a publication disclosing an invention two years prior to the filing of an application of a second inventor having a similar type of construction would

eliminate the second applicant in an interference between the claims filed by him and the broader claims of the first applicant. This is one of the advantages of lack of secrecy regarding an invention, though the publication after one year may bar the inventor from protection in a few countries, such as Germany, etc.

ARTICLE 7.

Care Regarding Correspondence.

Correspondence regarding an invention prior to issuance of papers should be carefully kept. The writer, for example, received a letter from a Milwaukee party requesting information regarding what countries he had patented a type of construction and he good-naturedly replied, failing by oversight to mention Canada as application pending. About a year later an interference was declared in the Canadian patent office on twenty-three claims between this same party and the writer and this correspondence was, of course, of immediate value.

ARTICLE 8.

Some Fundamental Difference in Laws of Different Countries.

The Canadian patent office practice is more nearly like that of the United States than any other country. In the United States, if a single claim is invalidated by decision of the court and the balance of the claims sustained, the patent is in force to that extent, while in England an adverse decision on one claim invalidates the whole patent, hence, while in the United States it is desirable to secure as many and as broad claims as possible, under the ruling such as the writer understands to be in force in England, it is desirable to limit the claims in such a manner that there is no risk of invalidating the patent by claiming too much.

ARTICLE 9.

Continued Rejection of Claims.

Very frequently the inventor gets discouraged by the rejection of claims by the patent office examiner on a really meritorious invention. This in no wise means that the inventor is not

entitled to protection, but rather that in the view of the examiner in charge of this division the claims are not drawn in a manner which permits him to allow them in view of the prior state of the art.

Generally matters of that kind are best taken up through a personal conference between the inventor, his attorney and the examiner. Many misunderstandings and the feeling that fair treatment has not been accorded will be eliminated by such a procedure.

ARTICLE 10.

An Example of a Fairly Complete Specification and Claim.

As an example of the preparation of an application covering in a fairly complete manner the state of the art at the time of the application, the writer appends to this article a copy of the Canadian patent issued to him on flat slab and column type of construction. The mushroom system being a form of this construction which he prefers to use, but the state of the art permitting broad claims on a continuous flat plate and column construction.

ARTICLE 11.

Exercise of Due Diligence.

In the field of practical construction it should be noted by the constructor that if he makes any advance in the art and fails to properly protect himself he takes a chance that some other one may, either by a similar or equivalent improvement, avail himself of his legitimate right to protection, patent the improvement and in turn charge the original inventor royalty for the use of an idea which perhaps he may have originated prior to the second inventor who had exercised due diligence in legally protecting his rights, provided, of course, that the second inventor had applied for the protection within the limit prescribed by the law.

ARTICLE 12.

Ethics of Securing a Patent by a Professional Engineer.

Attention has been called in the first part of the article to the object of the protection granted inventor by the patent laws.

Without this protection, a monopoly for a limited period, no originator of an invention of merit would be able to undertake the expensive pioneer work of introducing his meritorious improvement. Those who have peculiar notions regarding ethics of patent law and the propriety of the engineer availing himself of his rights in this respect have as a rule given this aspect of the question very little consideration.

ARTICLE 13.

Imitations of a New Device.

No invention of value is placed upon the market without almost immediately inviting a swarm of imitators who manage to change what they think is a slight detail and apply for a patent.

The old inventor never worries about this phase of the case; if the first in the field and he has drawn a broad and substantial foundation in the matter of claims upon which to proceed, he complacently smiles at the infringements and bides the proper time for presentation of his bill through the federal court.

In prosecuting infringements it is more profitable to move slowly so that the decision in the federal court if favorable will enable collection on a number of cases.

ARTICLE 14.

Transfer of Patents, Patent Rights and the Sale of Inventions.

Many otherwise bright business men purchase blue sky in this line. For example, one of the brightest business firms the writer is acquainted with paid a neat sum for exclusive rights in existing patents on a certain invention, including rights for that territory in anything he might invent and made themselves safe on neither end of the deal.

A right in a patent or invention can only be transferred by an instrument in writing designating the patent number, or if an application, the serial number, and preferably the subject-matter of the same. Then it should be properly recorded in the patent office to protect the right of the purchaser, otherwise after the interval of some three months the inventor, if dishonest, might sell the same rights to a second party, and if the sec-

ond party records the transfer before the first party his claim is secure. License, territorial rights, etc., should preferably be similarly recorded. The instrument of conveyance should be drawn up in such form that it may be made a matter of record, and the usual published forms should be preferably followed.

If in the regular employ of a company and the remuneration involves a specific agreement covering the product that the employee is employed to work out such an agreement may hold; though the indefinite transfer of what a man may invent at some future date is of no value.

ARTICLE 15.

What Constitutes a Patentable Invention in the Line of Building Construction.

We have noted that the patent laws of various countries have been devised as a method of encouraging advancement and development of the mechanical arts, and while the rules and practice of the United States Patent Office give a clear conception of many sides of this question, there are many in the building lines who have an exceedingly hazy idea as to what constitutes patentable invention in the line of construction work.

In general, invention may be defined as a combination or putting together of certain elements in a combination to produce a new result or new type of building construction, the combination and arrangement of elements in a frame which secures a useful result in economy of material or construction in a new and novel manner previously unknown and used. It may be only a new and economical wrinkle in methods of executing work. In general, invention involves new combinations of old elements, rather than those which are new or unknown.

As an example, the writer was granted a patent with thirteen claims on the Ferry Bridge at Duluth, Minn. In this structure there is not a single element which has not been used before for a different purpose. The combination, however, in the structure, forming a working mechanism for a specific purpose and combining the advantage of economy with stiffness of the traveler constituted a new and patentable invention. The suspension of the traveling frame on links, the rope drive, the support of the rope by swing hangers, the V-shaped point of the

traveler arranged to push aside the swing hangers, the pneumatic cushion receiving the car at the landing at the top of the car above, are each in themselves old and well known elements. Their combination in the working structure, however, forms a new and patentable novelty.

So in reinforced concrete we might take, for instance, the mushroom* system as an example. The column rods bending out horizontally are in themselves an old element, shown in the patent to Ellinger in beam construction. Diagonal reinforcement has been used by some of the earliest workers in the field. The combination, however, in a structure of flat slab floors and columns integral therewith and of the old elements of diagonal reinforcement, bent column rods, etc., constitute a new combination securing specific advantages from the standpoint of strength, economy and fireproof properties and hence the combination of the old elements in the new arrangement giving specific advantages forms the basis of a patentable invention.

The mere throwing together of steel or concrete in a heterogeneous hit or miss manner is unfortunately not clearly distinguished in our patent laws from the scientific combination of the elements to secure specific advantages in the commercial field.

The difference between a useful invention and the useless invention is, however, in general soon demonstrated by its adoption in the commercial field. Thus comparing the case of the mushroom system with that of one of the early German patents having a disc with rods hooked into the disc at the top of the column as a basis for a flat slab and column construction, the narrow width of belt of bars gives a coefficient of bending three or four times as great as that permissible with the mushroom system, leaving no opportunity for this class of construction to compete in the commercial field with the older types of beam and slab work.

Before issuance of patents to the writer in this line the criticism was common that there was nothing new in the element of diagonal reinforcement; that there was nothing new in the bending of the column rod horizontally into the concrete floor as it had been done in the beam construction; that there was nothing

*Six foreign patents now granted, ten others pending. Twenty-seven claims now allowed in United States, application which will issue shortly.

new in the general combination of concrete and steel and for that reason no patent would ever be granted for such a system.

In fact, it has been with some surprise that the writer has noted the tendency on the part of the examiners who are supposed to understand these principles clearly to select part of one patent containing an old feature and part of another patent containing an old feature and base a rejection on two old patents showing two old features illustrated therein as a reference for the rejection of a new combination involving the said old feature, though in the new combination they are used in a manner to secure a new result unattainable by the use of either alone or in combination with other old features.

Where, however, the new result may be obtained by the elimination of one element in an old combination the combination so claimed is broken and the originator of the new combination is entitled to protection in the use of the modification, hence in drawing the claims the importance of including only the necessary elements to secure the desired results becomes evident and should be kept clearly in mind by the inventor who desires to be protected in the use of the device or type of construction which he may have originated.

ARTICLE 16.

Strength and Weakness of Patents Dependent on How They Are Drawn.

Unfortunately many patents are not what they purport to be, due to defective claims and lack of skill in prosecuting the application. For illustration, take the Buffington patent on skeleton building construction. Quite a few of our Minneapolis attorneys supposed this broad enough to enable the patentee to collect royalty on this type of construction. While Mr. Buffington may not perhaps be credited with originating the idea, since it was unquestionably of a natural growth from the column and lintel, still he might even at that period have had secured claims upon which a respectable contest could have been made had it been differently handled.

Each claim, unfortunately, contained the element of a laminated column, *i. e.*, one made up of solid plates riveted together. Now such a column would be greatly lacking in economy and so impracticable to build that no one ever attempted it. The

elimination of this element in each combination claim left the patentee no opportunity to win.

This principle may be stated as follows: A claims and is allowed a combination claim of four elements to secure a given result. Later B secures the same result, using three elements used by A in a different manner and has broken the combination claimed by A.

Many have an idea that a patent amounts to very little and this view is correct if the patent is not properly prosecuted or if the state of the art does not allow the inventor to secure broad claims. Where, however, the case is properly prosecuted and the state of the art is such as to allow the claims to be broad in scope the patent serves as a basis for the strongest monopoly in the interest of the inventor, as has been proved again and again.

The above is presented not as a substitute for proper legal advice to the builder, but merely as a word of explanation along a line closely allied to the interests of the builder and owner, and if it may serve to answer some of the questions to others the writer has been frequently asked on this phase of right of inventor and others it has served its purpose.

SPECIFICATION

To All Whom It May Concern:

Be it known that I, Claude A. P. Turner, of the city of Minneapolis, county of Hennepin, state of Minnesota, United States of America, Civil Engineer, have invented certain new and useful improvements in **Steel Skeleton Concrete Construction,** and I do hereby declare that the following is a full, clear and exact description of the said invention, reference being had to the accompanying drawings, in which—

Figure 1 is a fragmentary side elevation of a reinforced column and floor slab, constructed in accordance with my invention, the reinforcement being shown by dotted lines;

Figure 2 is a detail thereof, being a cross section of Figure 1 taken on the line 2-2;

Figure 3, a top or plan view of the column reinforcement, and portions of the floor slab reinforcement belonging therewith;

Figure 4, another fragmentary side elevation of a reinforced column and floor slab, constructed in accordance with my invention, but of modified construction, the reinforcement also being shown by dotted lines;

Figure 5, a detail thereof, being a cross section of Figure 4 taken on the line 5-5;

Figure 6, a top of plan view of the column reinforcement and portions of the floor slab reinforcement belonging therewith;

Figure 7, still another fragmentary side elevation of a reinforced column and floor slab, constructed in accordance with my invention, but of another modified construction, the reinforcement being likewise shown by dotted lines;

Figure 8, a detail being a cross section of Fig. 7, taken on the line 8-8;

Figure 9, a top plan view of the column reinforcement, and of the floor slab reinforcement belonging therewith;

Figure 10, another fragmentary side elevation of a column and floor slab, constructed in accordance with my invention, but of modified construction, the reinforcement likewise shown by dotted lines;

Figure 11, a detail thereof, being a cross section of Figure 10 taken on the line 11-11;

Figure 12, a top or plan view of the column reinforcement and of portions of the floor slab reinforcement belonging therewith;

Figures 13 and 14, top or plan views of the major part of the floor slab reinforcement.

My invention relates to buildings or structures erected of reinforced concrete, and the object of my invention is to provide a column and slab or floor construction, requiring a minimum of concrete and reinforcement, but having all necessary strength, to the end that the cost of erection, both in respect to material and time required, may be substantially reduced and to secure certain important advantages which result from the structure which may be produced in accordance with my invention.

Similar letters refer to similar parts throughout the several views, A being the column, and B the floor slab.

In the practical application of my inventions, I vary the construction somewhat, as the exigencies of the case may require, adopting some one of the modifications herein shown and hereinafter described, or

combinations of two or more of them; for example, when the column reinforcement consists of the banded and bound vertical rods **a**, as shown in Figures 1, 2 and 3, I bend the said rods **a** outward at the top of the column A, and extend the laterally bent portions **a′** thereof horizontally, as shown in Figure 3; but where the column reinforcement consists of the banded and bound bars **b**, of structural steel, as shown in Figures 4, 5, 6, 7, 8 and 9 (bars too rigid to be practically bent), I employ the elbow ribs **c′** of which I arrange within the column A, and the horizontal portions **c2** of which I arrange within the floor slab B, as shown in Figures 4 and 7, which elbow ribs thereby become parts of both the column and the floor slab reinforcement.

Upon the horizontal portions **a′** of the vertical rods **a** constituting the column reinforcement shown in Figures 1, 2 and 3, upon the horizontal portions **c2** of the elbow ribs **c**, shown in Figures 4, 5, 6, 7, 8, 9 and 13, or upon the band **d** at the top of the reinforcement of the column A, I arrange the carrying rings **e** shown in Figures 3, 6 and 13, or the transverse bars **f** shown in Figures 9, 12 and 14, I arrange the direct, transverse and diagonal rods **j** shown in Figures 12, 13 and 14, which rods constitute the major portion of my floor slab reinforcement.

A description in detail of each form of construction herein shown is as follows:

By reference to Figures 1, 2 and 3 of the drawing, it will be seen that the column reinforcement consists of a number of vertical rods, **a** (usually eight), which rods are arranged in a circle, the diameter of which is nearly as large as that of the column to be moulded thereupon. These vertical rods (**a**) united at intervals intermediate their length, by means of the bands **d**, said bands being secured to the vertical rods **a**, by means of U-shaped bows or yokes **h**, which bows or yokes are secured thereto by means of the clamping nuts **i**. These vertical rods **a** I bend outwardly at the top of the column A, and arrange the laterally bent portions **a′** thereof radially as shown in Figure 3, which laterally bent portions extend into, and constitute a part of the floor slab reinforcement. Upon the radially arranged portions **a′** of the column reinforcement **a**, I place the carrying rings **a**, shown in Figures 3, 6 and 13, or the carrying bars **f**, shown in Figures 7, 9 and 12 and 14, upon which rings or bars in turn, I lay the direct, transverse and diagonal rods **j**, shown in Figures 12, 13 and 14, which rods constitute the major portion of the floor slab reinforcement.

By reference to Figures 4, 5 and 6 of the drawing, it will be seen that the column reinforcement consists of a group or series of angle bars **k** (structural steel), which bars (like the vertical rods **a**) united at intervals intermediate their length by means of the transverse tie-plates **l**, which tie-plates are arranged in two series, such alternate plate constituting one of the said series, and each alternate plate constituting the other one of the said series, the first named series being arranged transversely to the last named series.

These plates (**l**) are secured to the vertical bars **k** by means of the rivets **n**, and the vertical bars **k** are bound at their upper ends by means of the band **d**.

In this construction I employ the elbow ribs **c**, arranging their vertical portions **c′** within the band **d**, and among the vertical bars **k**, which constitute the major part of the column reinforcement, and the horizontal portions of **c2** of which I arrange radially, as shown in Fig. 6.

Upon the radially arranged portions **c2** of the elbow ribs **c**, I lay the direct, transverse and diagonal rods **j** shown in Figures 12, 13 and 14, which rods constitute the major part of the floor slab reinforcement.

By referring to Figures 7, 8 and 9 of the drawing, it will be seen that, while the column reinforcement consists of vertical bars of structural steel, the construction differs somewhat from the construction shown in Figures 4, 5 and 6, inasmuch as the column reinforcement con-

sists of three bars **o**, of structural steel, I-shaped in cross section, called usually I-beams.

In this construction (as in the construction shown in Figures 4, 5 and 6), I employ ribs **c**, above described, and arranged thereupon the carrying rings **e**, shown in Figures 3, 5 and 13, or other equivalents carrying bars **f** shown in Figures 9, 12 and 14.

In this construction, I perforate the upper portions of the vertical bars **o**, and pass one or more of the carrying bars **f** through the said vertical bars **o**, as shown in Figures 7 and 9, thereby binding a portion of the reinforcement of the floor slab B, to the reinforcement of the column A.

In this construction the elbow ribs **c** may be dispensed with and the direct, transverse and diagonal reinforcing rods **j** be laid directly upon the carrying bars **f**.

This construction is practically adapted for use in the quarter-round supports **p** at the corners of the room, and the half-round supports **q**, lying between them, which supports are adapted to receive the ends of the direct, transverse and diagonal rods **j** where the same abut the walls of a room or building.

By referring to Figures 10, 11 and 12 of the drawing, it will be seen, that while the column reinforcement consists of banded and bound rods, it differs from the construction shown in Figures 1, 2 and 3, inasmuch as that I do not bend the vertical rods constituting the column reinforcement horizontally, to constitute a part of the floor slab reinforcement, as in the first construction described.

In this modification, I preferably employ eight vertical rods **r** arranged in the order shown in the two parallel series of three members each, and a pair of rods arranged between the said parallel series, all of the eight rods **r** being bound at their upper portions and at intervals intermediate their length by means of the bands **s**, and four of the rods **r** being similarly bound, by means of the bands **s′**, laying diagonally to the first named band **s**. Upon the upper bands **s** and **s′**, I arrange the transverse rods of bars **f** shown in Figures 12 and 14, upon which bars or rods I arrange the direct, transverse and diagonal rods **j** shown in Figures 13 and 14.

Any or all of these forms relate to and properly belong to my invention, inasmuch as they have been suggested to my mind while in the capacity of an engineer I have been supervising the erection of structures of considerable magnitude in this and other countries.

By my construction, wherein a capital or enlargement is formed on the column, or in one piece therewith, and by the employment of my reinforcement, I am able to dispense with the use of beams on the under side of the floor slab. This is an immense advantage in every way. It is economical in the use of concrete; it is also economical in that it renders unnecessary the expensive forms for making the beams, and it means greater rapidity of work. As far as the finished structure is concerned, the absence of beams on the under side of the floor slab enables partitions to be placed anywhere that it may be found desirable to place them. It results in better illumination from the windows, and there are no dirt-collecting corners, which exist where beams or girders are employed.

Another very important advantage resulting from the provision of a ceiling that is smooth, or free from beams or projections, is in the matter of fire protection. In fighting a fire with a stream of water from a hose, the obstruction offered by ribs or beams is obviously serious, since a rib may stop short a stream of water, whereas a flat, smooth surface against which the stream is directed at an angle, will deflect and spread the water, causing it to descend to the floor over a wide area and to the best possible advantage. Where sprinkler heads are used in a ceiling, the cost of equipment by such a system of fire pro-

tection is substantially reduced, because fewer sprinkler heads are required with a flat or smooth ceiling than one where there are beams or ribs on the under side of the ceiling. In warehouses or similar buildings, my invention is of special value because in order to afford aisles or passageways, the load is naturally concentrated around the columns, and it is at these points, where the load therefore is greatest, that the greatest strength of the structure exists, by reason of the enlarged capitals of the columns, and their integral construction or formation with the slabs, and the heavy reinforcements of the structure immediately at and adjacent to the column. The provision of the capitals on the columns by gradually increasing the diameter of the columns at the top, and making them and the slab an integral mass, takes care of the compression of the concrete which is the greatest over the columns.

As will be seen by reference to the drawings, the rods forming the framework or head at the top of the column extend laterally into the slab substantial distances beyond the sides of the column, and as the rods are anchored in or supported at their inner ends by the column structure, the arrangement is that of a cantilever, so that I avail myself of a cantilever for supporting the slab. It will be observed that there is a concentration of the rods at each column, and the rods extend or radiate from the point of concentration at the column into the floor slab.

Having thus described my invention, what I claim is:

1. In a monolithic concrete structure, the combination of a concrete slab having a smooth ceiling or under surface, and columns of reinforced concrete having capitals integral with the columns and integral with the slab, and directly supporting the slab.

2. In a monolithic concrete structure, the combination of a concrete slab having a smooth ceiling or under surface, columns of reinforced concrete having capitals integral with the columns and integral with the slab, and directly supporting the slab, and reinforcements extending from the columns into the slab.

3. In a monolithic concrete structure, the combination of a concrete slab having a smooth ceiling or under surface, columns of concrete having capitals integral with the columns and integral with the slab, and directly supporting the slab, reinforcing rods extending vertically through the columns, and horizontally from the upper ends thereof into the slab, and horizontally extending rods in the slab supported by the column-reinforcing rods.

4. The combination of a concrete slab, columns of concrete, reinforcing rods extending vertically through the columns, said rods having members arranged in a circular series and that extended radially therefrom into the slab, and rods in the slab which extend crosswise of said laterally extending members of the column reinforcing rods.

5. The combination of a concrete slab, concrete columns, vertically disposed members constituting column reinforcement, horizontally disposed members constituting slab reinforcement, and supplemental reinforcing members extending in a circular series over the upper portion of the columns and laterally into adjacent portions of the slab.

6. In a monolithic concrete structure, the combination of a concrete slab, having a smooth ceiling or under surface, columns of reinforced concrete having capitals integral with the columns and integral with the slab, and directly supporting the slab, and groups of rods running through the slab from column to column, a group of rods in a transverse direction being of a width equal or substantially equal to the diameter of their supporting means.

7. In a monolithic concrete structure, the combination of a concrete slab having a smooth ceiling or under surface, columns of reinforced concrete having capitals integral with the columns and integral with the slab, and directly supporting the slab, a group of rods extending in

different directions at the top of the column, and outward therefrom, and groups of rods running from column to column through the slab and overlapping said rods at the top of the column.

8. In a steel skeleton concrete construction, the combination of a vertically reinforced column and a floor supported thereby, the vertical reinforcing rods of the said column being bent laterally outward and extended radially into said floor, carrying rods resting upon and supported by the horizontal portions of said laterally bent rods, and horizontal rods in said floor arranged upon said carrying rods, said horizontal rods extending directly from column to column substantially as shown and described.

9. In steel skeleton concrete construction, the combination of a vertically reinforced column, and a floor slab supported thereby, said column reinforcing rods being bent laterally and extending into the said floor slab, carrying rods arranged upon the laterally bent portions of said column reinforcement, and the floor slab reinforcing rods arranged upon the said carrying rods and extending directly and diagonally from column to column as shown.

10. In a reinforced concrete floor, the combination of a vertically reinforced column having vertical reinforcing rods bent laterally into the floor slab, carrying rods arranged on the laterally bent portions of said rods, and direct and diagonal reinforcement in slabs running from column to column.

11. A monolithic structure comprising an integral mass or body of concrete forming columns with capitals and a horizontal slab on the under side of which the capitals directly adjoin, and reinforcements radiating from the columns into the slab.

12. The combination of a concrete floor slab, and a vertically reinforced column having vertical reinforcing rods bent laterally into the floor slab, carrying rods arranged on the laterally bent portions of said rods, and direct and diagonal reinforcement in the slabs running from column to column.

13. The combination of a vertically reinforced concrete column, and a concrete floor supported thereby, the vertical reinforcing rods of the said column being bent laterally outward and extended radially into the said slab, carrying rods resting upon and supported by the horizontal portions of the said laterally bent rods, and horizontal rods in the said floor arranged upon the said carrying rods, said horizontal rods extending directly from column to column, substantially as shown and described.

14. The combination of concrete columns, a concrete floor supported thereby, having a smooth ceiling or under surface, vertically extending reinforcing members in the columns, reinforcing members that extend in a circular series concentric with the columns and into the floor at the top of the column, horizontally extending reinforcing members in the floor supported by said circular series of reinforcing members, and extending therefrom into the floor.

15. The combination of a concrete column having a series of vertical rods, means connecting said rods and forming a framework, a framework at the top of the column, extending laterally outward in different directions, and a concrete slab supported by the column, and integral with the column.

16. The combination of a concrete column comprising a series of vertical rods, means connecting said rods and forming a framework therewith, a framework at the top of the column extending laterally outward in different directions and consisting of a series of radial members and concentric circular members supported by the radial members, and a concrete slab supported by the column and integral with the column.

17. The combination of a reinforced concrete column, an open framework forming a head at the top of the column, and a concrete slab

supported by the column and integral therewith, said open framework projecting laterally outward in different directions into and terminating in the slab and anchored in the column structure.

18. In a monolithic concrete structure, the combination of a concrete slab having a smooth ceiling or under surface, columns of concrete, reinforcing rods extending vertically through the columns, and horizontally from the upper ends thereof into the slab, and horizontally extending rods in the slab supported by the column reinforcing rods.

19. In a reinforced concrete structure, the combination with a plurality of separate concrete columns, and the vertical reinforcement therein, of a set of reinforcing members embedded in a column and principally supported thereby, the ends of said set of reinforcing members being separated so as to radiate from the column support into the adjacent floor slab to substantially all parts thereof, and a concrete floor slab embedded and supported by said radiated ends.

20. In a reinforced concrete structure, a concrete floor slab having a plurality of sets of reinforcing members embedded therein, the said reinforcing members to each set being concentrated at one point in their length and principally supported at or near the point of concentration while the ends of said reinforcing members are separate so as to diverge from the point of concentration into various portions of the floor slab.

21. A monolithic structure composed of concrete, comprising columns and a horizontal slab moulded into an integral body, and having a cantilever supporting system of reinforcement extending from the columns into the slab and supported by the columns, and direct and diagonal reinforcement overlapping said cantilever system.

22. In a reinforced concrete structure, the combination of a concrete slab, reinforced concrete columns having rods extending laterally into the slab in different directions forming a cantilever supporting system around the column at the floor level, and groups of rods supported by said laterally extended rods and extending through the slab.

23. In a reinforced concrete structure, the combination of a concrete slab free from ribs, reinforced concrete columns having rods extending laterally from a column into the slab and forming a cantilever supporting system, and groups of rods supported by the latter and extending through the slab in various directions, the width of a group of rods being substantially equal to the diameter of their supporting means.

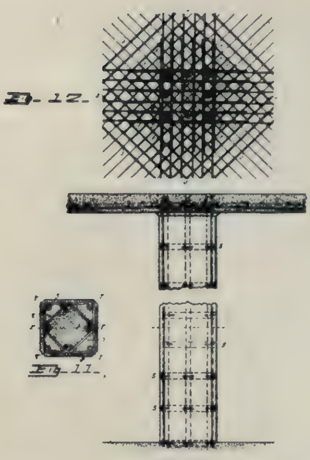


Fig. 12. Fig. 11. Fig. 10.

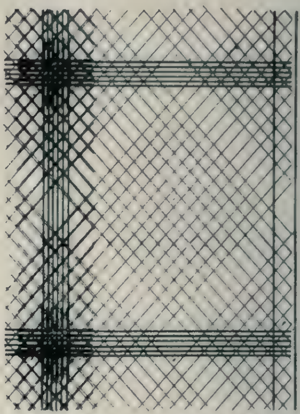


Fig. 14.

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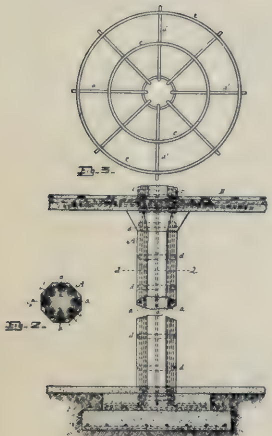

Fig. 3. Fig. 2. Fig. 1.

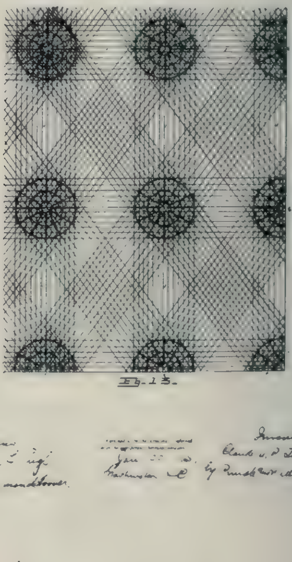

Fig. 13.

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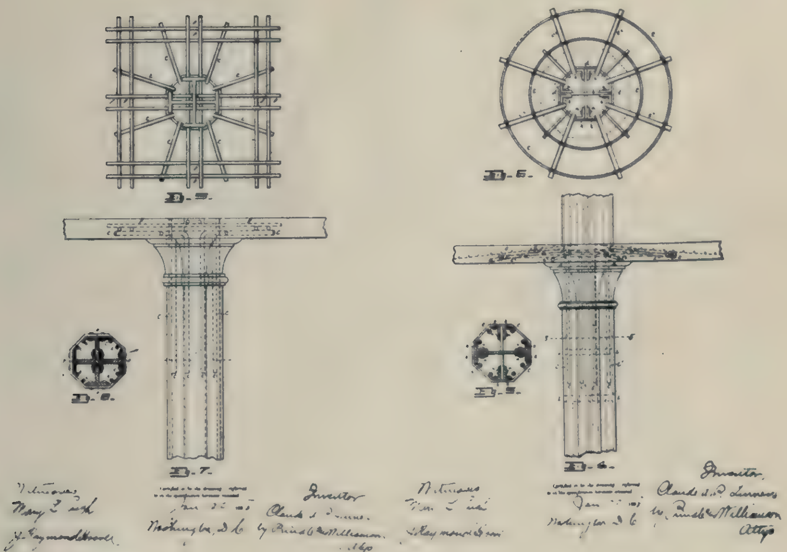
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